DYNAMIC RESPONSE OF PEATS

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16. ABSTRACT

The objectives of this project were to investigate the dynamic behavior of peat and to evaluate the influence of that behavior on the dynamic response of peat deposits. During the course of the research, the objectives were expanded to explore the effects of strong earthquake shaking on pile foundations embedded in peat deposits.

An experimental investigation was undertaken using samples of a peat deposit typical of many large peat deposits in western Washington. This deposit, known locally as Mercer Slough in Bellevue, Washington, is crossed by a series of bridges that form Interstate 90 and its collector-distributor ramps. Test specimens were obtained by careful, undisturbed sampling of the Mercer Slough peat. The specimens were tested in a specially modified resonant column device.

The resonant column tests showed that the peat was very soft and that its stiffness increased with effective confining pressure in a manner similar to that displayed by cohesionless soils. The peat exhibited nonlinear stress-strain behavior, and the degree of nonlinearity was observed to decrease with increasing effective confining pressure. Damping was observed to increase with increasing shear strain, but at a rate that decreased with increasing effective confining pressure. Special tests designed to investigate frequency dependence of peat stiffness and damping showed that the degree of frequency dependence was relatively small.

A series of ground response analyses was performed to investigate the seismic response of peat deposits. Three vertical profiles through Mercer Slough were analyzed using both equivalent linear and nonlinear ground response analyses. The analyses showed that the soft nature of the Mercer Slough peat would produce amplification of the long-period components of an earthquake ground motion. Substantial long-period motions can produce large dynamic displacements that are potentially damaging to bridges and bridge foundations. Comparison of the equivalent linear and nonlinear ground response analyses showed that the equivalent linear model tended to oversoften the peat at low effective confining pressures, leading to underprediction of ground motion amplitudes at low periods.

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Dyn Reponse of Peats

DYNAMIC RESPONSE OF PEATS

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EXECUTIVE SUMMARY

The historical glaciation of western Washington is responsible for the formation of numerous peat deposits. Many of these deposits are substantial, reaching thicknesses of 20 m or more. A number of peat deposits lie in important transportation corridors and are crossed by highway bridges. Because western Washington is a seismically active area, the long-term safety of these bridges will depend on the seismic response of the peat deposits they cross. The existing geotechnical earthquake engineering literature, however, contains virtually no information on the seismic response of peat deposits.

This report describes a research investigation of the dynamic properties of peat and the dynamic response of peat deposits. An experimental investigation was undertaken using samples of a peat deposit typical of many large peat deposits in western Washington. This deposit, known locally as Mercer Slough in Bellevue, Washington, is crossed by a series of bridges that form Interstate 90 and its collector-distributor ramps. Test specimens were obtained by careful, undisturbed sampling of the Mercer Slough peat. The specimens were tested in a specially modified resonant column device.

The resonant column tests showed that the peat was very soft and that its stiffness increased with effective confining pressure in a manner similar to that displayed by cohesionless soils. The peat exhibited nonlinear stress-strain behavior, and the degree of nonlinearity was observed to decrease with increasing effective confining pressure. Damping was observed to increase with increasing shear strain, but at a rate that decreased with increasing effective confining pressure. Special tests designed to investigate frequency dependence of peat stiffness and damping showed that the degree of frequency dependence was relatively small.

A series of ground response analyses was performed to investigate the seismic response of peat deposits. Three vertical profiles through Mercer Slough were analyzed using both equivalent linear and nonlinear ground response analyses. The analyses showed that the soft nature of the Mercer Slough peat would produce amplification of the long-period components of an earthquake ground motion. Substantial long-period motions can produce large dynamic displacements that are potentially damaging to bridges and bridge foundations. Comparison of the equivalent linear and nonlinear ground response analyses showed that the equivalent linear model tended to oversoften the peat at low effective confining pressures, leading to underprediction of ground motion amplitudes at low periods.

The research produced valuable information on an important topic for which virtually no information was previously available. The results of this research will allow characterization of the dynamic response of peats with much greater reliability than was previously possible. It will also allow more accurate ground response analyses to be performed. The complexity of the peat and the limited number of tests that could be performed in this investigation indicate that additional site-specific investigations should be performed for the design of significant new structures that cross peat deposits. The establishment of strong motion accelerometer stations in and near a large peat deposit would provide extraordinarily useful data on the seismic response of these materials and on the seismic vulnerability of bridges that cross them.

INTRODUCTION AND RESEARCH APPROACH

THE PROBLEM

The historical occurrence of strong earthquakes in western Washington is well established. Such earthquakes can subject transportation structures to severe dynamic loading. Knowledge of the nature of this loading is required for earthquake-resistant design of new structures and for evaluation of the seismic vulnerability of existing structures. Because seismic loading is strongly influenced by soil conditions, knowledge of the dynamic behavior of the soil that supports a particular structure is required.

The landforms of western Washington are strongly influenced by historical glaciation. The retreat of the most recent glaciers left many depressions that were subsequently filled with organic matter. This organic matter has decayed in place to form peat deposits. Some of these deposits cover substantial areas and are quite thick. Mercer Slough in Bellevue, for example, covers an area of several square kilometers with peat that is over 18 m thick. Interstate 90 crosses Mercer Slough with four parallel bridges supported on piles that extend through the peat to stiffer underlying soil. The seismic performance of those and other bridges in similar environments is of great interest.

The dynamic behavior of peat is poorly understood. Considerable uncertainty exists regarding the stiffness, damping, and degradation characteristics of peat under cyclic loading conditions. To accurately evaluate the seismic vulnerability of structures that cross peat deposits, WSDOT engineers require improved characterization of the dynamic behavior of peat.

RESEARCH OBJECTIVES

The objectives of this project were to investigate the dynamic behavior of peat and to evaluate the influence of that behavior on the dynamic response of peat deposits. Realization of these objectives involved a comprehensive program of dynamic laboratory testing, characterization of dynamic peat properties, and adaptation of existing analytical tools for computation of the seismic response of peat deposits. During the course of the research, the objectives were expanded to explore the effects of strong earthquake shaking on pile foundations embedded in peat deposits.

BACKGROUND

The response of geologic materials to dynamic loading has been of interest to geotechnical engineers for many years. Considerable progress has been made in understanding and characterizing dynamic soil properties. Historically, most research has focused on the dynamic response of inorganic soils, although some soft clays with significant organic content have also been studied. Recently, interest has developed in the dynamic response of highly organic deposits such as peat. The origin of much of this interest stems from the need to evaluate the seismic stability of pile-supported highway bridges crossing substantial peat deposits. The failure of the Struve Slough bridge (Housner et al., 1990) near Watsonville, California, in the 1989 Loma Prieta earthquake illustrated the potential vulnerability of such bridges to earthquake-induced damage. The need for evaluation of the seismic stability of embankments constructed on peat, such as those in the extensive levee system of the Sacramento River delta area of California, provides additional motivation for studies of the dynamic response of peat.

Dynamic Soil Behavior

Earthquake ground motions may produce dynamic shear stresses in soil deposits; the nature of these stresses may be broadly characterized by their amplitude, frequency content, and duration. These characteristics are different for different earthquakes. When soils are subjected to dynamic loading, their response is governed by their density, stiffness, and damping characteristics. Whereas the density of a soil is virtually independent of ground motion characteristics, ground motions may be strongly influenced by soil stiffness and damping characteristics.

Effect of Amplitude

The effect of ground motion amplitude on soil properties is generally expressed in terms of shear strain amplitude. The variation of soil stiffness, generally expressed in terms of shear modulus, with cyclic shear strain amplitude has been studied by numerous investigators (Hardin and Black, 1968; Seed and Idriss, 1970b, Hardin and Drnevich, 1972a, 1972b). More recently, the reduction of shear modulus with increasing cyclic shear strain has been found to depend on soil plasticity (Kokoshu et al., 1982; Dobry and Vucetic, 1990; Sun et al., 1988; Vucetic and Dobry, 1991). Damping ratio is known to increase with increasing cyclic shear strain amplitude (Hardin, 1965; Seed and Idriss, 1970b), and recent studies (Kokoshu et al., 1982; Dobry and Vucetic, 1990; Sun et al., 1988; Vucetic and Dobry, 1991) have indicated that this variation is also influenced by soil plasticity. As illustrated in Figure 1, soils of high plasticity behave more linearly and with lower damping than soils of low plasticity.

Effect of Frequency Content

In a theoretical analysis of the dynamic response of a poro-elastic medium, Biot (1956) showed that the stiffness and damping characteristics of a saturated soil were, because of viscous losses caused by relative movement of the pore water with respect to the soil skeleton, dependent on the frequency of loading. Ishihara (1970) showed that, for most soils, the range of frequencies at which variation of stiffness and damping is significant is much greater than those commonly of interest to geotechnical engineers.

Dynamic tests (Hardin, 1965, Hardin and Scott, 1966) have shown that the stiffness and damping ratio of dry sand is insensitive to loading frequency. These results have been confirmed by others, and it is common to assume that the damping ratio of cohesionless soil is independent of loading frequency. The influence of loading frequency on the damping characteristics of cohesive soils is less certain. Most experimental investigations in which the effects of loading frequency on dynamic response have been reported have interpreted the effects in terms of the number of stress-controlled loading cycles required to achieve a particular level of cyclic strain; consequently, the results cannot be interpreted in terms of shear modulus and damping ratio. Thiers and Seed (1969) showed that the number of cycles to failure in San Francisco Bay mud was strongly influenced by doubling the loading frequency from 1 Hz to 2 Hz, while others have found little effect for the same variation of loading frequency on different clays. Wood (1982), in a review of cyclic testing of soils, stated that it "seems likely that the effect of frequency is greater for more plastic clays, and intuitively it seems reasonable that slower cycles give the clay, as a viscous material, more time to follow the applied load and are likely to produce greater strains and greater pore pressures." In practice, however, the stiffness and damping characteristics of cohesive soils are also usually assumed to be independent of loading frequency.

Effect of Duration

The influence of ground motion duration on soil properties has received relatively little attention outside the realm of liquefaction analysis. Soil stiffness is known to degrade (Thiers and Seed, 1969; Idriss et al., 1978; Vucetic, 1990) with increasing numbers of stress or strain cycles. The influence of duration on damping characteristics has not been explicitly reported.

Peat

The term "peat" has been used both correctly and incorrectly to describe many different materials. The most commonly used classification systems use ash content to identify peat. The ash content is generally obtained by firing an oven-dried sample in a muffle furnace until the soil has been reduced to an inorganic ash. The temperature and duration of firing vary; ASTM D2794 (ASTM, 1991) recommends that a temperature of 550^{o}C be held until the material is completely ash. Generally, soils with ash contents of less than 25 percent are classified as peat (Andrejko et al., 1983), although others have suggested that an ash content of 20 percent be used (Landva et al., 1983).

Peat has long been recognized by geotechnical engineers as a problematic material. It is usually noted for its low unit weight, low shear strength, high compressibility, and rate-dependent behavior (Landva and LaRochelle, 1983; Marachi et al., 1983). The viscous nature of peat behavior gives rise to phenomena such as secondary compression, creep, and stress relaxation. This aspect of peat behavior leads to its frequent characterization as a viscous fluid for problems in which long-term deformation is important.

Previous Work

Very little research has been performed on the dynamic response of peat. The only such work identified in the literature was associated with the foundation investigation for a proposed (but never constructed) highway tunnel in the Union Bay area of Seattle, Washington (Shannon &

Wilson, 1967). The Union Bay site, also located on Lake Washington but about 11 km northwest of Mercer Slough, consisted of up to 18 m of peat underlain by up to 24 m of soft to medium stiff clay. The clay was underlain by heavily overconsolidated, dense glacial till. In 1966, water contents in the Union Bay peat ranged from 700 percent to 1,500 percent at the surface, with a trend of slightly decreasing water content with increasing depth. Saturated unit weights ranged from 1.003 Mg/m^3 to 1.058 Mg/m^3 and averaged 1.021 Mg/m^3. Atterberg limits tests showed liquid limits of 700 to 1,000 and plasticity indices of 200 to 600. The Union Bay and Mercer Slough peats were deposited under similar conditions, and it is expected that their important geotechnical characteristics are also similar.

The 1967 foundation investigation, conducted under the direction of Stanley D. Wilson, included a number of interesting and innovative laboratory and field experiments. In the laboratory, samples of the Union Bay peat were subjected to repeated load triaxial tests. Loading was apparently applied pneumatically through a solenoid-controlled valve, resulting in a loading pattern that was characterized as a triangular wave with a period of 0.5 seconds. In the field experiments, various dynamic tests were conducted near a set of three seismometers installed in a boring in Union Bay. The upper seismometer was located about 3.3 m below the surface of the 17.4-m-thick Union Bay peat deposit. The second seismometer was located 18.6 m deep in the 12.5-m-thick silty clay layer immediately below the peat. The lowest seismometer was installed 32 m deep in the very dense glacial till beneath the clay. The water level at the location of the seismometers was approximately 1 m to 2 m above the surface of the peat. In the dynamic tests, accelerations were measured in response to a number of sources of dynamic loading that included forced and free lateral vibrations of adjacent piles, driving of a nearby pile into the till, blasting with dynamite at different distances from the seismometers, natural microtremors, a distant nuclear blast, and a magnitude 4.5 earthquake at an epicentral distance of about 40 km and focal depth of 24 km to 32 km. Each of these loading sources produced motions of different amplitude, frequency content, and duration. On the basis of the results of these laboratory and field tests, Shannon and Wilson (1967) concluded that "the damping factor in the peat is quite high, of the order of 20 percent or more, particularly in shear. Qualitative tests in the laboratory indicate even higher damping factors ..."

Tsai (1969) used the recorded motions from the magnitude 4.5 earthquake in an early study of the influence of local soil conditions on earthquake ground motions. The recording of the seismometer in the peat deposit, however, was neglected in this approximate analysis with the explanation that "with the unusually high water content ... the peat media would behave essentially like a viscous fluid and can hardly sustain shear wave motions."

Seed and Idriss (1970a) analyzed the recorded earthquake motions using an equivalent linear, lumped mass technique (Idriss and Seed, 1970). The motion measured in the glacial till was applied as a rigid base motion (Idriss, personal communication, 1991) at the clay/till interface. Shear moduli were obtained from shear wave velocity measurements and repeated loading tests. Damping ratios were estimated with the reasoning that "because of [the peat's] fibrous and less cohesive characteristics, damping for peat would be expected to be higher than for clay." The damping ratios used in the analyses were approximately three times as large as those used at the time for clay, ranging from approximately 8 percent at a cyclic shear strain of 0.00005 percent to approximately 19 percent at a cyclic shear strain of 0.03 percent. With these damping ratios, reasonable agreement was obtained between the measured accelerations and the accelerations predicted by the rigid base model. The modulus reduction and damping curves used for peat in the Seed and Idriss study are shown in Figure 2. These curves suggest that peat exhibits considerably greater nonlinearity and damping than typical inorganic soil.

Kramer (1993) reported the results of cyclic triaxial and piezoelectric bender element tests on Mercer Slough peat specimens. The specimens tested in that investigation were obtained by pushing Shelby tubes in a rotary wash boring that extended through a 5-m-thick parking lot fill that had been placed on top of Mercer Slough peat. The specimens were consolidated to effective confining pressures of 10 to 30 kPa, with an average effective confining pressure of 10 kPa. Average modulus reduction and damping behavior from that laboratory investigation are shown in

Figure 3. These tests indicated behavior that was more linear than that reported by Seed and Idriss (1970a), but they also revealed unusual damping behavior.

APPROACH

Mercer Slough

The peat specimens tested in this investigation were obtained from Mercer Slough. Mercer Slough is a peat-filled extension of Lake Washington in Bellevue, Washington (Figure 4). The surface of the slough is flat and heavily overgrown with horsetails, grasses, and small trees. Lake Washington water levels are maintained at nearly constant level at the Hiram Chittenden Locks in Seattle; consequently, the groundwater level in Mercer Slough is generally within 30 cm of the ground surface.

The thickness of the Mercer Slough peat is variable across the slough, with a maximum thickness of approximately 18 m along the alignment of Interstate 90, which crosses Mercer Slough by means of four pile-supported bridge structures. While some of the deepest peat has been dated at more than 13,000 years old, most was deposited since the isolation of Lake Washington from Puget Sound by the Cedar River alluvial fan some 7,000 years ago (Newman, 1983). Since that time the peat appears to have accumulated at an average rate of approximately 0.175 cm/yr. The peat is underlain by very soft to medium stiff silty clay and occasional loose to dense sand, which is in turn underlain by heavily overconsolidated, dense glacial till. Tertiary bedrock in the area is found at about 305 m deep. A subsurface profile along the I-90 alignment is shown in Figure 5.

The Mercer Slough peat is fibrous at shallow depths and becomes less fibrous and more highly decomposed with increasing depth. In recent subsurface investigations, water contents were generally between approximately 500 percent and 1,200 percent, with no apparent trend with depth. A limited number of vane shear tests indicated peak undrained strengths ranging from 0.75 kPa to 8 kPa. Piezocone penetration tests showed a uniform tip resistance of approximately 17.0 kPa, with a friction ratio of 1.5 percent to 6.0 percent. Pore pressures during penetration were essentially hydrostatic. However, interpretation of small-scale insitu tests, such as the vane shear test and cone penetration test, has been recognized as being very difficult in peat (Landva, 1986). A small number of UU triaxial tests produced undrained strengths of 2.5 kPa to 8.8 kPa, with very large strains required for full strength mobilization. Interpretation of the results of a large-scale lateral pile load test suggested a cohesive strength of approximately 10 kPa (Kramer et al., 1990). Significant time-dependent response, in the form of creep and stress relaxation, was observed during the performance of those tests.

For this investigation, a series of peat specimens were obtained from a single boring near the center of Mercer Slough close to Interstate 90. As indicated in the boring log of Figure 6, the site was underlain by 18 m of peat. Drilling and sampling of peat is quite difficult; the boring was advanced by rotary wash techniques using a lightweight, portable tripod, and samples were obtained by pushing Shelby tubes. The boring log indicates that considerable difficulty was encountered in sampling the peat. Of 28 attempted samples, 18 produced no sample recovery, and only incomplete recovery was obtained in most of the remaining samples. Many of those samples showed visible evidence of disturbance. At a later date, additional samples of the Mercer Slough peat were obtained as part of an investigation by Converse Consultants NW of the seismic vulnerability of a Seattle Water Department pipeline that crosses Mercer Slough. These samples, obtained using a piston sampler with sharpened Shelby tubes in rotary wash borings, were made available to the University of Washington for resonant column testing. These samples showed no visible evidence of disturbance.

Experimental Investigation

The researchers performed 21 resonant column tests. The tests were divided into two series. The first series (10 tests) followed a conventional resonant column test procedure: the loading frequency was slowly increased until the response reached its maximum value (at the fundamental frequency), then the specimen was placed into free vibration. The fundamental frequency was used to compute the shear modulus, and the free vibration attenuation behavior was used to compute the damping ratio. The second series of tests was performed with a very sensitive miniature torque cell mounted between the resonant column loading head and the specimen. The torque cell allowed direct measurement of torque-rotation behavior from which stress-strain curves could be obtained. In this series of tests, torque-rotation behavior was measured over a wide range of frequencies, including the fundamental frequency. Free vibration tests were also performed as in the first series of tests.

FINDINGS

The majority of the resonant column tests were performed on normally consolidated specimens. Tests were performed over wide ranges of effective confining pressures, strains, and loading frequencies. Four pairs of tests were conducted on specimens from the same sampling tube; in these tests one specimen was normally consolidated and the other was overconsolidated.

EFFECT OF CONFINING PRESSURE

Near the ground surface, normally consolidated Mercer Slough peat is extremely soft and weak. It is so soft, in fact, that it is virtually incapable of propagating shear waves. As a result, it is reasonable to assume that normally consolidated Mercer Slough peat has zero shear wave velocity (and, consequently, zero shear modulus) at zero effective confining pressure. This type of behavior is also exhibited by cohesionless inorganic soils such as clean sands and gravels. The maximum shear modulus of the Mercer Slough peat was observed to increase with increasing effective confining pressure. At the lowest effective confining pressures used in this investigation (approximately 1.5 kPa), the maximum shear moduli were very low. The low-strain stiffnesses of normally consolidated test specimens increased with effective confining pressure, but in an irregular pattern (Figure 7).

EFFECT OF STRAIN AMPLITUDE

The Mercer Slough peat exhibited a pronounced degree of nonlinear stress-strain behavior. As a result, the shear modulus decreased sharply with increasing shear strain amplitude. Modulus reduction curves for individual resonant column tests on normally consolidated specimens displayed generally similar shapes, although more scatter was observed than is common for similar tests on clay or sand specimens. The modulus reduction data for all tests on normally consolidated specimens are shown together in Figure 8. The test data indicated that the peat behaved essentially linearly at shear strains of up to about 0.001 percent, but that the stiffness decreased relatively quickly thereafter. The rate of modulus reduction appeared to be influenced by effective confining pressure. Although all of the resonant column tests were performed at relatively low effective confining pressures and produced somewhat scattered modulus reduction results, there was a distinct trend of increasing linearity with increasing effective confining pressure. This trend is consistent with the behavior observed in the previous cyclic triaxial tests (Kramer et al., 1990), as illustrated in Figure 9. This trend is also consistent with modulus reduction behavior observed in tests on a peat from the eastern United States at much higher effective confining pressures (K. Stokoe, personal communication, 1996). The test data also showed that the peat retained a higher portion of its original stiffness at very large shear strains (greater than 1 percent) than typical inorganic soils.

The Mercer Slough peat also exhibited inelastic behavior. Damping characteristics measured by free vibration tests showed that the peat had a high damping ratio at low strain levels (Figure 10). Although laboratory-measured damping data are frequently scattered, the Mercer Slough test data suggested that the damping ratio (at a particular strain level) tends to decrease with increasing effective confining pressure. Because of the apparent effects of sampling disturbance, this trend could not be corroborated with the damping measurements from the cyclic triaxial tests of Kramer et al. (1990), but results of tests at higher effective confining pressures on a peat from the eastern United States are consistent with this behavior (K. Stokoe, personal communication, 1996), as shown in Figure 11.

EFFECT OF LOADING FREQUENCY

The time-dependent behavior of Mercer Slough peat under long-term loading conditions has been well established. Creep of the Mercer Slough peat adjacent to a roadway fill caused substantial vertical and lateral displacements that required frequent maintenance and eventual repair. Significant creep and stress relaxation behavior has also been observed in lateral pile load tests (Kramer, 1991); the lateral load on a pile held at constant displacement typically dropped by 10 percent in the first minute after loading. Such behavior implies that the peat could exhibit high damping at very low loading frequencies.

The addition of the torque transducer allowed measurement of individual hysteresis loops over a wide range of loading frequencies in Tests 11 through 20. These hysteresis loops helped determine shear moduli and damping ratios at different loading frequencies and strain levels during the frequency sweeps that were used to identify the fundamental frequencies of the resonant column specimens. Because the strain level could not be controlled in these tests, frequency-dependence was examined by interpolating individual test results as necessary to strain levels of 3 x 10^{-4} percent, 1 x 10^{-3} percent, and 1 x 10^{-2} percent.

The test results showed that loading frequency had a modest effect on shear modulus at all three strain levels. The damping ratio was also modestly influenced by loading frequencies, at least at frequencies below about 10 Hz. Damping ratios at frequencies above 10 Hz, however, increased dramatically. The high damping ratios at high frequencies may help explain the difficulty of measuring insitu shear wave velocities in impact-type tests over large distances in peat.

EFFECT OF OVERCONSOLIDATION

The results of tests on pairs of specimens considered to be virtually identical except for overconsolidation ratio were inconclusive. No significant trends in the positions or shapes of the shear modulus, modulus reduction, or damping curves were evident.

CONCLUSIONS

The results of the resonant column tests can be interpreted in a relatively straightforward manner. To put these results into context, it is useful to compare the behavior of the Mercer Slough peat with the behavior of other types of soil that have been extensively investigated.

EFFECT OF CONFINING PRESSURE

The data suggested that the variation of maximum shear modulus with effective confining pressure can be approximated by a relationship developed for cohesionless soils (Seed and Idriss, 1970b)

$$G_{max} = 1000\, K_{2,max} (\sigma'_m)^{0.5} \qquad (1)$$

where σ'_m is the mean principal effective stress in psf. As illustrated in Figure 12, $K_{2,max}$ appears to fall in the range of 1 to 3. For comparison, sands generally exhibit $K_{2,max}$ values that range from about 30 (for loose sands) to 70 (for dense sands). Obviously, Mercer Slough peat is considerably softer than even the loosest of sands.

The limited available data suggested that the modulus reduction behavior of Mercer Slough peat is also influenced by effective confining pressure. Specifically, the Mercer Slough peat exhibited more linear behavior and lower damping at higher effective confining pressures.

EFFECT OF STRAIN LEVEL

Figure 13 shows how the dynamic properties of Mercer Slough peat compare with those assumed for Union Bay peat by Seed and Idriss (1970a). Mercer Slough peat behaved far more linearly (i.e., with constant shear modulus) at shear strains below about 0.1 percent than was assumed for Union Bay peat. At higher strains, modulus reduction values for the two peats agree well. The postulated damping characteristics of Union Bay peat correspond to the upper range of the measured damping characteristics of Mercer Slough peat.

The dynamic characteristics of Mercer Slough peat can also be compared with those of inorganic soils. Figure 14 illustrates the modulus reduction and damping characteristics of Mercer Slough peat and those of inorganic soils of different plasticity (Vucetic and Dobry, 1991). At shear strains of less than about 0.1 percent, the average modulus reduction behavior was similar to that observed for nonplastic (PI = 0) inorganic soils by Vucetic and Dobry (1991). At higher strains, however, the peat retained much more of its initial stiffness than a nonplastic inorganic soil. This aspect of the peat behavior is thought to be related to the fibrous nature of the peat; individual fibers tend to straighten and stretch at higher strain levels and provide a mechanism of shearing resistance not available in particulate soils. However, Mercer Slough peat exhibited considerably greater damping at low strain levels than inorganic soils. At high strain levels (above about 0.5 percent), the damping ratio of Mercer Slough peat appears to be similar to those of inorganic soils with PI < 50.

COMPARISON WITH PREVIOUS MERCER SLOUGH TEST RESULTS

These test results were compared with those from cyclic triaxial tests reported by Kramer (1993). As previously described, the cyclic triaxial tests were performed on samples obtained by pushing Shelby tubes from a WSDOT drill rig. Because the samples were obtained from an area overlain by 5 m of fill, the cyclic triaxial tests were performed at higher effective confining pressures than the resonant column tests of the present investigation.

Maximum shear moduli from the previous investigation are compared with those from the present investigation in Figure 15. The maximum shear moduli are considerably lower than those that were measured in the present investigation. The differences in maximum shear moduli are attributed to sampling disturbance. Sampling by simply pushing Shelby tubes causes considerably more disturbance than piston sampling. This disturbance tends to influence low-strain properties, such as maximum shear modulus and low-strain damping ratio, more than properties at higher strains.

Modulus reduction behavior from the previous and present investigations are shown in Figure 16(a). The peat appeared to behave more linearly in the previous investigation, as evidenced by its flatter and higher modulus reduction curve. This aspect of the peat behavior may result from the higher effective confining pressures used in the previous tests. However, it must also be recognized that the specimens tested in the previous investigation were influenced by sampling disturbance. It is possible that a reduction in low-strain stiffness due to disturbance had the effect of flattening and raising the modulus reduction curve for the previous investigation.

The effects of sampling disturbance in the previous investigation are most profoundly displayed in the damping behavior illustrated in Figure 16(b). The specimens tested in the previous investigation exhibited unusual behavior—damping ratios were extremely high (greater than 40 percent) at low strain levels and dropped to more typical values at high strain levels. The specimens tested in the present investigation exhibited more conventional damping behavior. The extremely high damping behavior observed at low strain levels in the previous investigation is attributed to sampling disturbance that damaged the low-strain "structure" of the peat.

EFFECT ON SEISMIC GROUND RESPONSE

The dynamic properties of Mercer Slough peat were used to evaluate the nature of the ground surface motions that would be expected in a strong earthquake near Mercer Slough. The ground response analyses were performed for three of the soil profiles investigated by Kramer (1993). The profile identification nomenclature used by Kramer (1993) is also used in this report. The characteristics of the three profiles are described in Table 1.

Table 1. Soil Profiles Used in Ground Response Analyses

Layer	A	B	E
Fill	2.4 m	0 m	0 m
Peat	3.4 m	9.1 m	18 m
Soft Clay	0 m	0 m	12 m
Dense Sand	299 m	296 m	274 m

The properties assigned to the fill (γ_m = 1.60 Mg/m^3; $K_{2,max}$ = 52), soft clay (γ_{sat} = 171 Mg/m^3; PI = 20), and dense sand (γ_{sat} = 2.28 Mg/m^3; $K_{2,max}$ = 80) were identical to those used in the previous investigation. The maximum shear modulus of the peat was calculated from Equation 1 using $K_{2,max}$ = 2 and γ_{sat} = 1.06 Mg/m^3. Modulus reduction and damping behavior was represented by the average values of the ranges of behavior shown in figures 8 and 10.

The three profiles were subjected to two input motions—the scaled Lake Hughes motion used in the previous investigation and a bedrock motion developed from a probabilistic seismic hazard analysis (PSHA) conducted as part of a seismic hazard vulnerability evaluation of the Alaskan Way Viaduct (Kramer et al., 1995). The two input motions are illustrated in Figure 17. In both cases, the input motions were applied at bedrock located approximately 305 m below the surface of Mercer Slough.

Equivalent Linear Analyses

The Lake Hughes motion allows evaluation of the effects of the improved dynamic peat properties on ground response. In the previous investigation, the peak ground surface accelerations caused by the Lake Hughes motion at profiles A, B, and E were 0.15 g, 0.10 g, and 0.08 g, respectively. As indicated in Figure 18, the corresponding peak accelerations from the present investigation were 0.20 g, 0.07 g, and 0.04 g. The peak acceleration of profile A increased by 33 percent, primarily because of the increased linearity and lower damping at the higher effective confining pressure induced by the fill that overlies the peat. The peak accelerations at profiles B and E, where effective confining pressures were much lower, were 30 percent to 50 percent lower than previously estimated. This reduction is attributed to the increased nonlinearity shown to exist at lower effective confining pressures. As would be expected, spectral accelerations (Figure 19) followed the same pattern. The previous ground response analyses produced peak spectral accelerations of 0.38 g, 0.31 g, and 0.20 g for profiles A, B, and E, respectively. The corresponding peak spectral accelerations from the present investigation were 0.89 g, 0.17 g, and 0.11 g.

The response to the PSHA-based bedrock motion was consistent with the response to the Lake Hughes motion, although the PSHA-based motion was stronger. As illustrated in Figure 20, the PSHA-based bedrock motion produced peak ground surface accelerations of 0.28 g, 0.09 g, and 0.06 g at profiles A, B, and E. Peak spectral accelerations (Figure 21) were 1.07 g, 0.25 g, and 0.16 g, respectively. Again, the strongest amplification was observed at profile A.

Nonlinear Analyses

Ground response analyses were repeated with a nonlinear ground response analysis program. This program allowed the use of nonlinear backbone curves that were consistent with the modulus reduction curves used in the equivalent linear analyses. Thus, differences in the results of the nonlinear and equivalent linear analyses are due solely to the assumption that nonlinear stress-strain behavior can be approximated by strain-compatible equivalent linear behavior.

Nonlinear analyses of profiles A, B, and E subjected to the Lake Hughes input motion produced the accelerograms shown in Figure 22 and the response spectra shown in Figure 23. Computed peak ground surface accelerations were 0.24 g, 0.17 g, and 0.23 g for profiles A, B, and E, respectively. These values were higher than those computed by the equivalent linear analyses, particularly for profiles B and E where the peat was thicker and softer. This apparently counterintuitive result can be explained by careful observation of the stress-strain behavior of the peat just above its contact with the underlying stiffer soil. In this region, the nonlinear model shows that the shear strain in the peat is relatively small except for a few cycles of motion; as a result, the peat exhibits a (relatively) high shear modulus for the great majority of the earthquake. The shear modulus of the peat in the equivalent linear analysis, however, is controlled by the peak shear strain. This causes the equivalent linear shear modulus to be quite low throughout the duration of the earthquake. The peat is thereby oversoftened in the equivalent linear analysis, and lower peak and spectral accelerations are predicted.

Similar trends were observed for the PSHA-based input motion (Figures 24 and 25). Computed peak ground surface accelerations for profiles A, B, and E were 0.31 g, 0.30 g, and

0.27 g, respectively. Again, both peak and spectral accelerations are greater than those predicted by the equivalent linear analysis, particularly for the softer profiles B and E.

EFFECT ON PILE FOUNDATIONS

To investigate the effects of the dynamic response of the Mercer Slough peat deposit on the stresses in piles extending through the peat, a dynamic pile-soil interaction analysis was conducted. The pile-soil interaction analysis was performed with a nonlinear, dynamic Winkler beam analysis (Horne, 1996) that includes both near- and far-field effects.

The analyses were performed for piles with two different diameters and flexural stiffnesses: a 30.5-cm-diameter pile with a flexural stiffness of 8.6×10^6 N-m^2 and a 61-cm-diameter pile with a flexural stiffness of 8.6×10^7 N-m^2. The piles were assumed to be 12 m long, and at Profile B, they were assumed to extend through 9.1 m of Mercer Slough peat and 2.9 m into the underlying dense sand. The computed ground motions from the SHAKE analysis of Profile B with the PSHA-based bedrock motion were used as input to the pile-soil interaction analysis.

The pile-soil interaction analysis indicated that the maximum pile bending moments occurred at approximately the contact between the peat deposit and the underlying dense sand. Bending moment profiles at 0.02-sec time increments are shown in Figures 26 and 27. The maximum bending moments for the 30.5-cm and 61-cm piles were 109,000 N-m and 266,000 N-m, respectively. Assuming that the piles were steel pipe piles, these maximum bending moments would produce maximum extreme fiber stresses of 393 MPa and 193 MPa, respectively. As a result, yielding would be expected in the 30.5-cm pile. Maximum bending moments and maximum extreme fiber stresses for piles of other flexural stiffnesses may be estimated using the maximum curvature values shown in Figure 28.

RECOMMENDATIONS AND IMPLEMENTATION

The investigation of the dynamic response of peat described in this report was limited, by availability of samples, to the peats of Mercer Slough in Bellevue, Washington. These results should be applicable to peats at other sites in western Washington that were deposited under similar geologic conditions, but that similarity should be verified at specific sites by field and/or laboratory testing. It should also be recognized that peats are inherently variable materials and that the results of a single investigation involving a limited number of laboratory tests are not sufficient to define the behavior of these complicated materials with great precision. Nevertheless, this research project has greatly improved understanding of the dynamic properties of peat.

The results of the research can be implemented by following several recommendations for characterizing dynamic peat properties during seismic ground response analyses. The specific recommendations are as follows:

1. Maximum shear moduli should be estimated, whenever possible, from shear wave velocities measured in the field using crosshole, downhole, SASW, or seismic cone penetration tests. At many peat sites, such testing may be quite difficult. In such cases, maximum shear modulus of Mercer Slough peat can be estimated using

$$G_{max} = 1000\ K_{2,max}\ (\sigma'_m)^{0.5}$$

   where σ'_m is the mean principal effective stress in psf, and $K_{2,max}$ varies between 1 and 3, with an average value of 2.

2. Soil nonlinearity can be expressed in terms of modulus reduction curves. The modulus reduction curves for Mercer Slough peat vary with effective confining pressure and can be estimated by interpolating between the modulus reduction curves shown in Figure 9 to the effective confining pressure of interest. For nonlinear analyses, nonlinear backbone curves can be computed directly from these modulus reduction curves.

3. The energy dissipation characteristics of soils can be expressed in terms of damping curves. The damping curves for Mercer Slough vary with effective confining pressure and can be estimated by interpolating between the damping curves shown in Figure 11 to the effective confining pressure of interest.
4. The low stiffness of peat deposits will cause amplification of the long-period components of an earthquake motion. If the long-period components of the input motion used in a ground response analysis of a peat deposit are in error, computed ground surface motions may have substantial errors. Ground response analyses of peat deposits should be performed with processed, baseline-corrected accelerograms.
5. The results of this research can be used to develop average amplification factors for peat deposits of different thicknesses in western Washington. These amplification factors may be useful for rapid estimation of ground motion levels in peat deposits.
6. For analysis and design of important structures, the variability and uncertainty of dynamic peat properties should be recognized and evaluated by means of sensitivity analyses.
7. The low strength and stiffness of peat deposits can lead to the development of large strains during earthquake shaking. Such conditions may lead to oversoftening of the peat in equivalent linear ground response analyses. For important structures that cross soft peat deposits, ground response should be computed by using nonlinear ground response analyses with backbone curves computed from the modulus reduction curves shown in Figure 9.
8. The establishment of strong motion accelerometers in and near a large peat deposit would provide invaluable supporting evidence for the results of this research investigation. A basic system would include two instruments—one on the surface of the peat and one at the bottom of the peat or at a nearby stiff soil outcrop. Additional instruments could be added to bridge structures in the vicinity to provide

information on structural response. Such a strong motion instrumentation package would greatly help improve characterization of seismic vulnerability, even from measurements taken in relatively small earthquakes; its cost would be a very small fraction of the damage (and liability) that could result from a strong earthquake.

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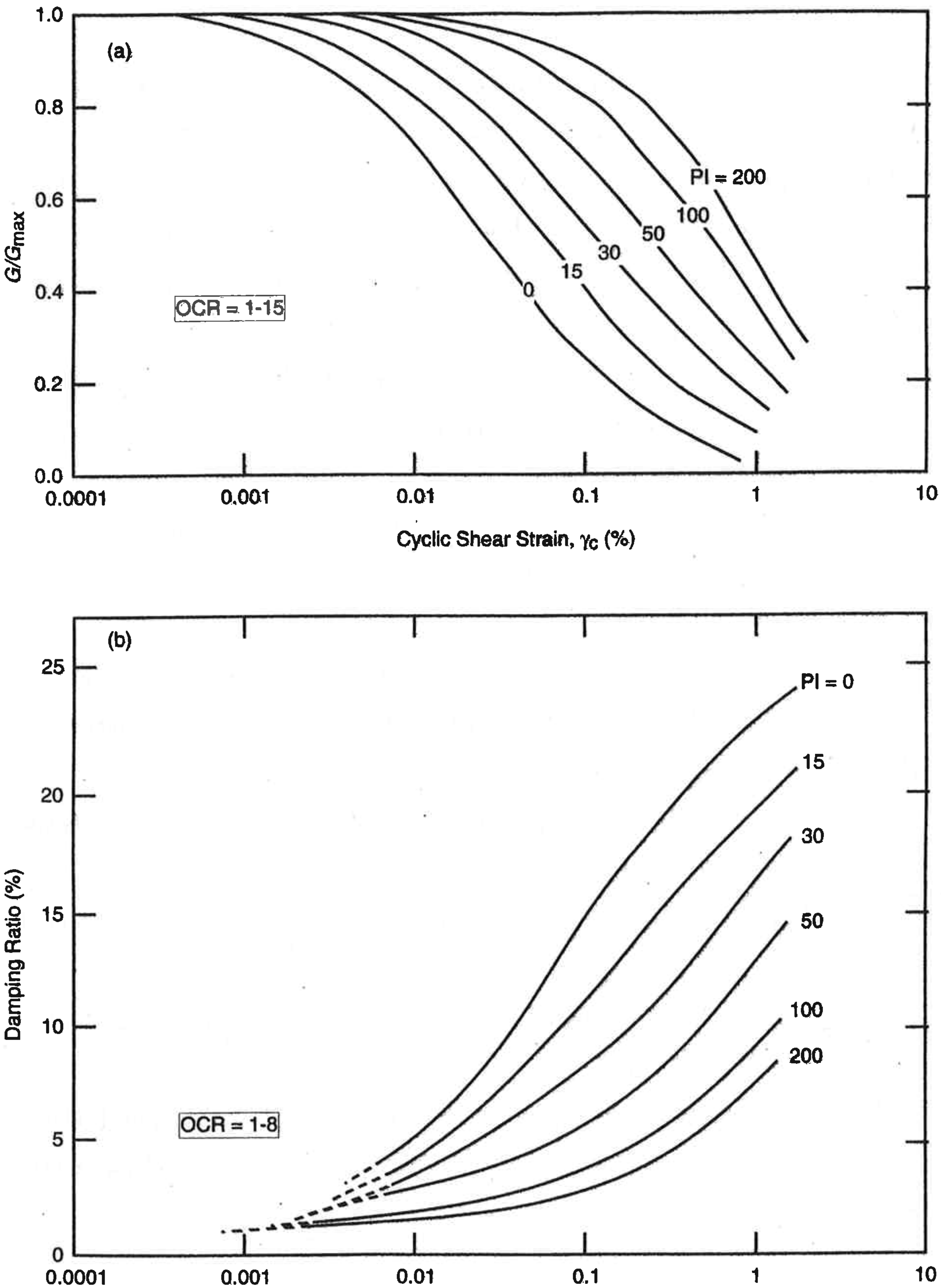


Figure 1. Variation of (a) Modulus Reduction and (b) Damping Behavior with Shear Strain and Plasticity Index for Inorganic Soils (after Vucetic and Dobry, 1991)

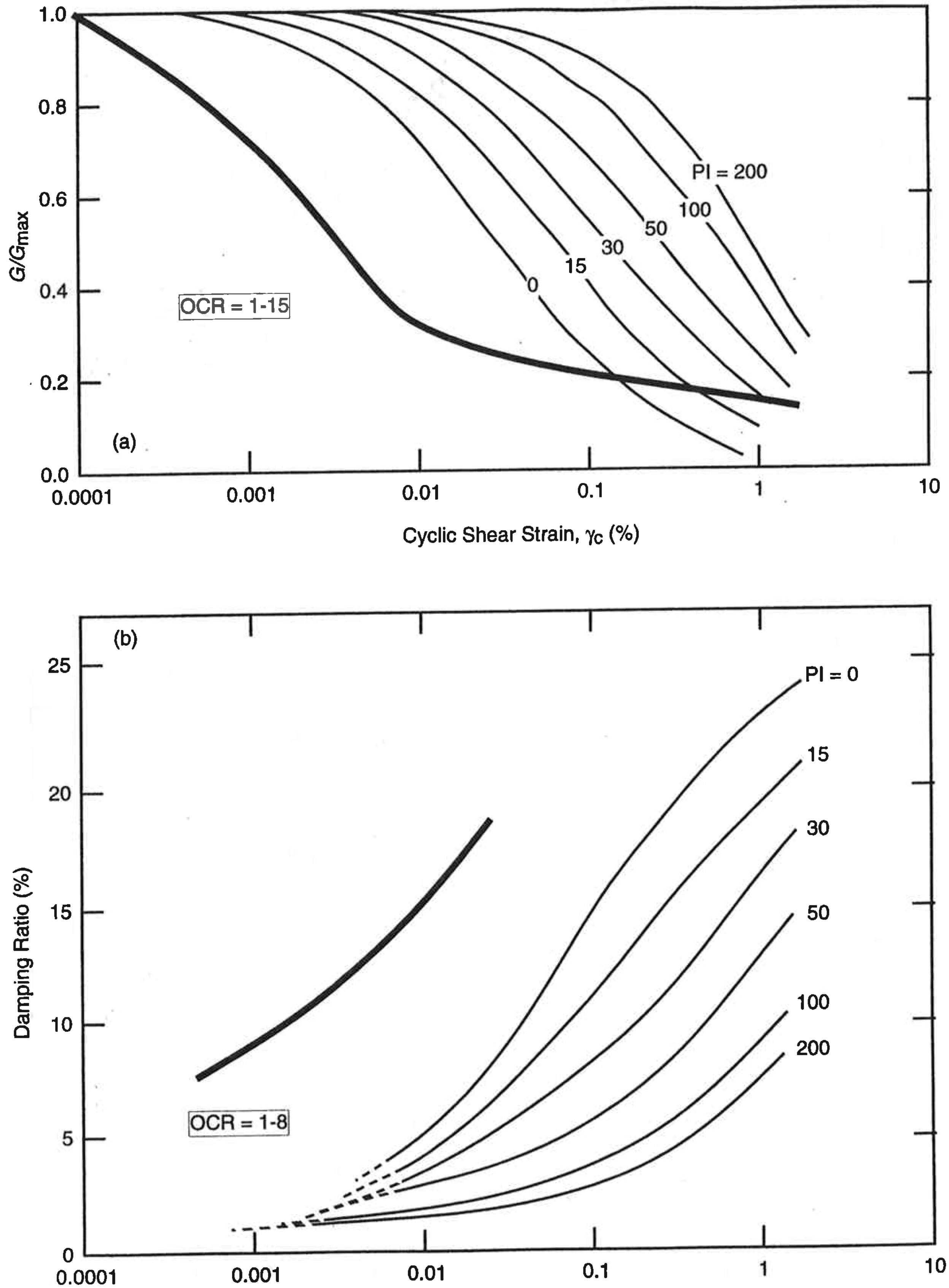
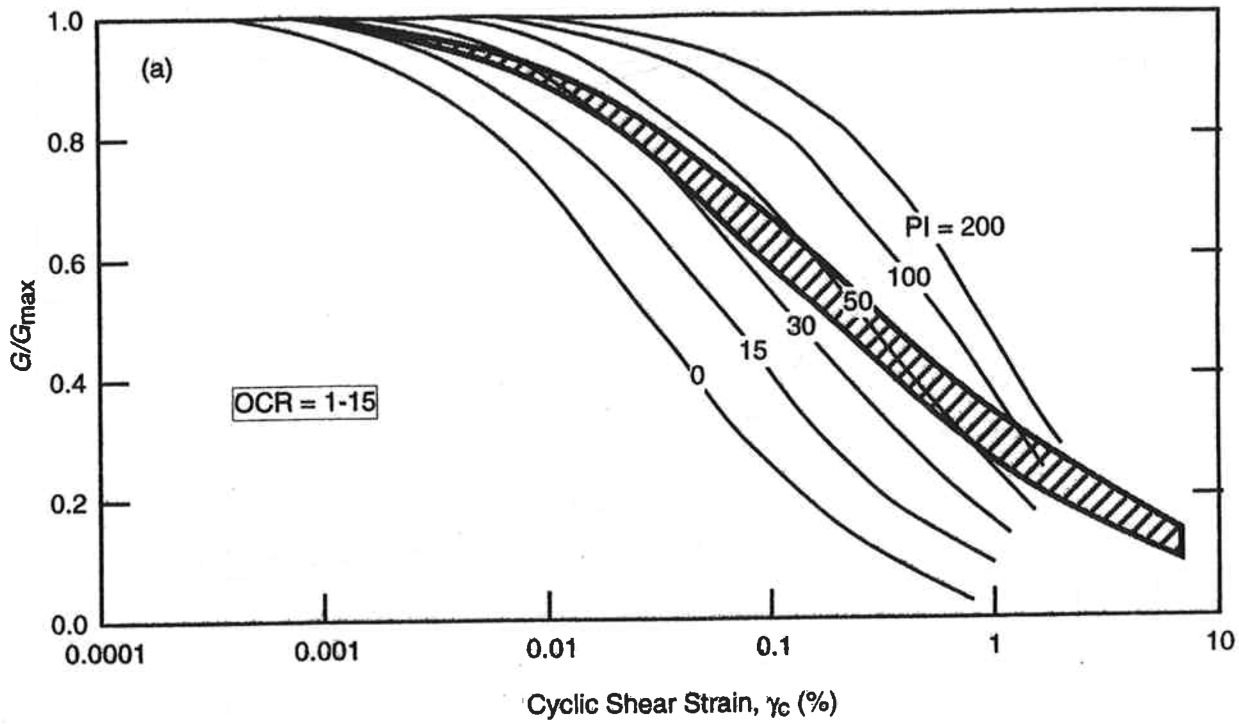


Figure 2. Estimated (a) Modulus Reduction and (b) Damping Behavior for Union Bay Peat (after Seed and Idriss, 1970a)



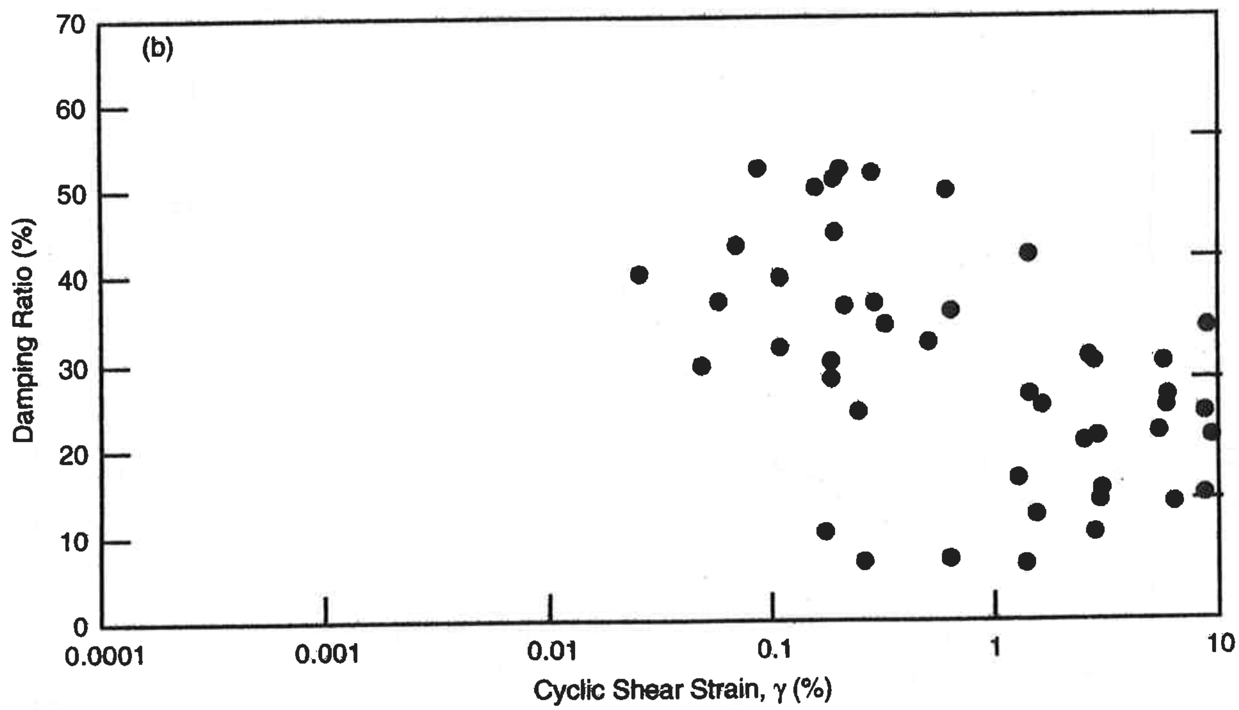


Figure 3. Estimated (a) Modulus Reduction and (b) Damping Behavior for Mercer Slough Peat from Piezoelectric Bender Element and Cyclic Triaxial Tests (after Kramer et al., 1991)

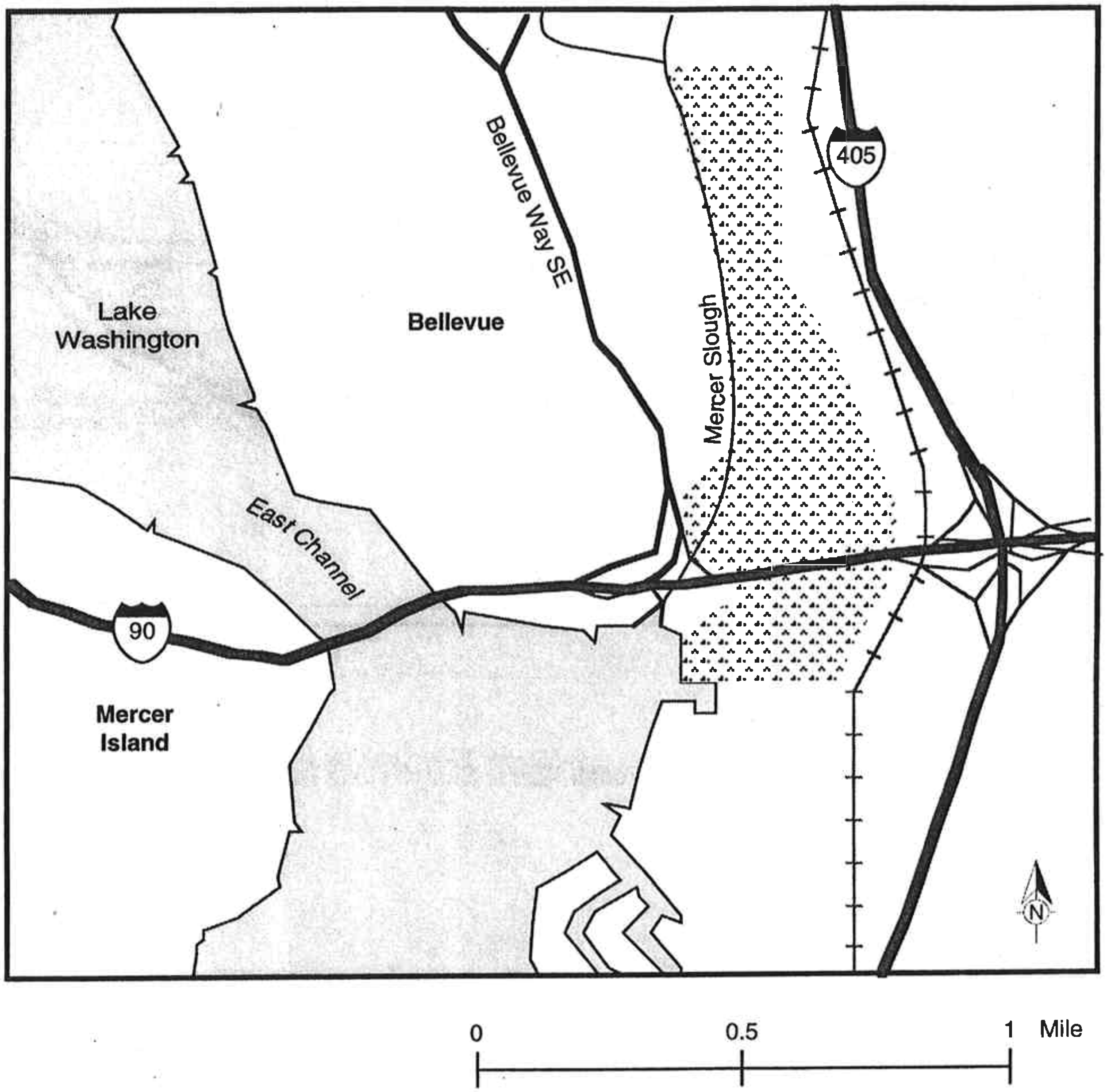


Figure 4. Location of Mercer Slough in Bellevue, Washington

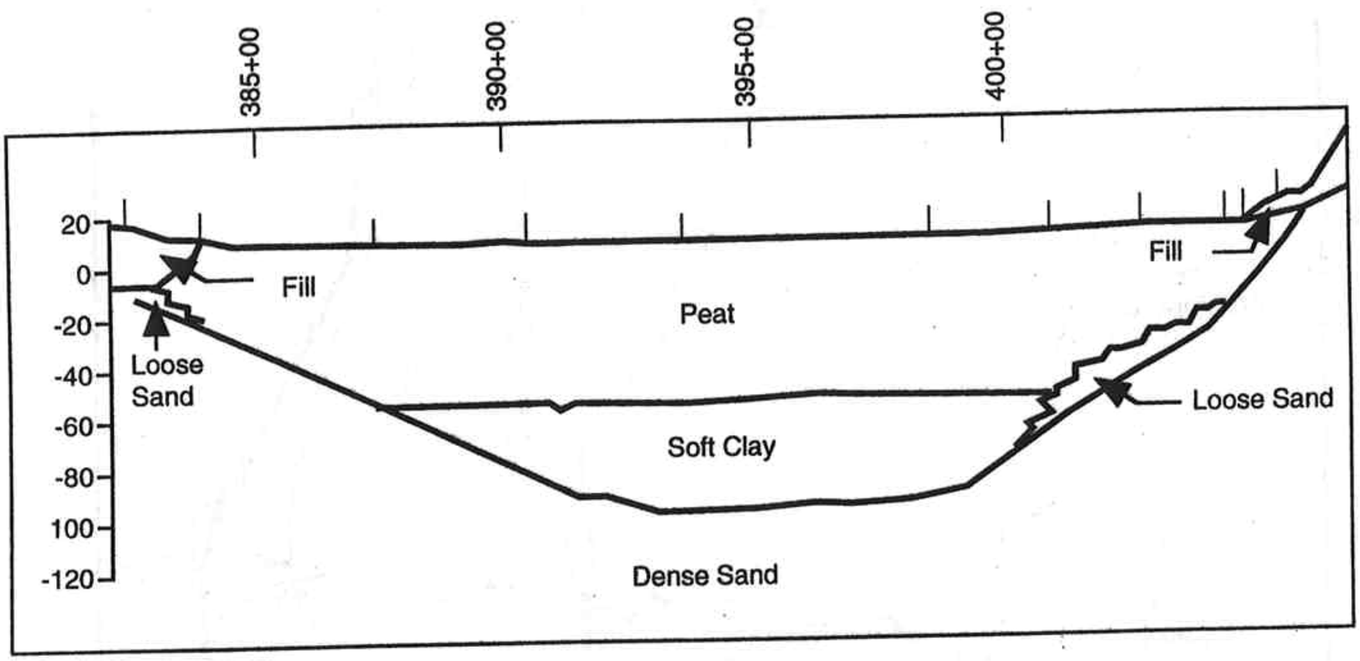


Figure 5. Subsurface Profile through Mercer Slough along I-90 Alignment

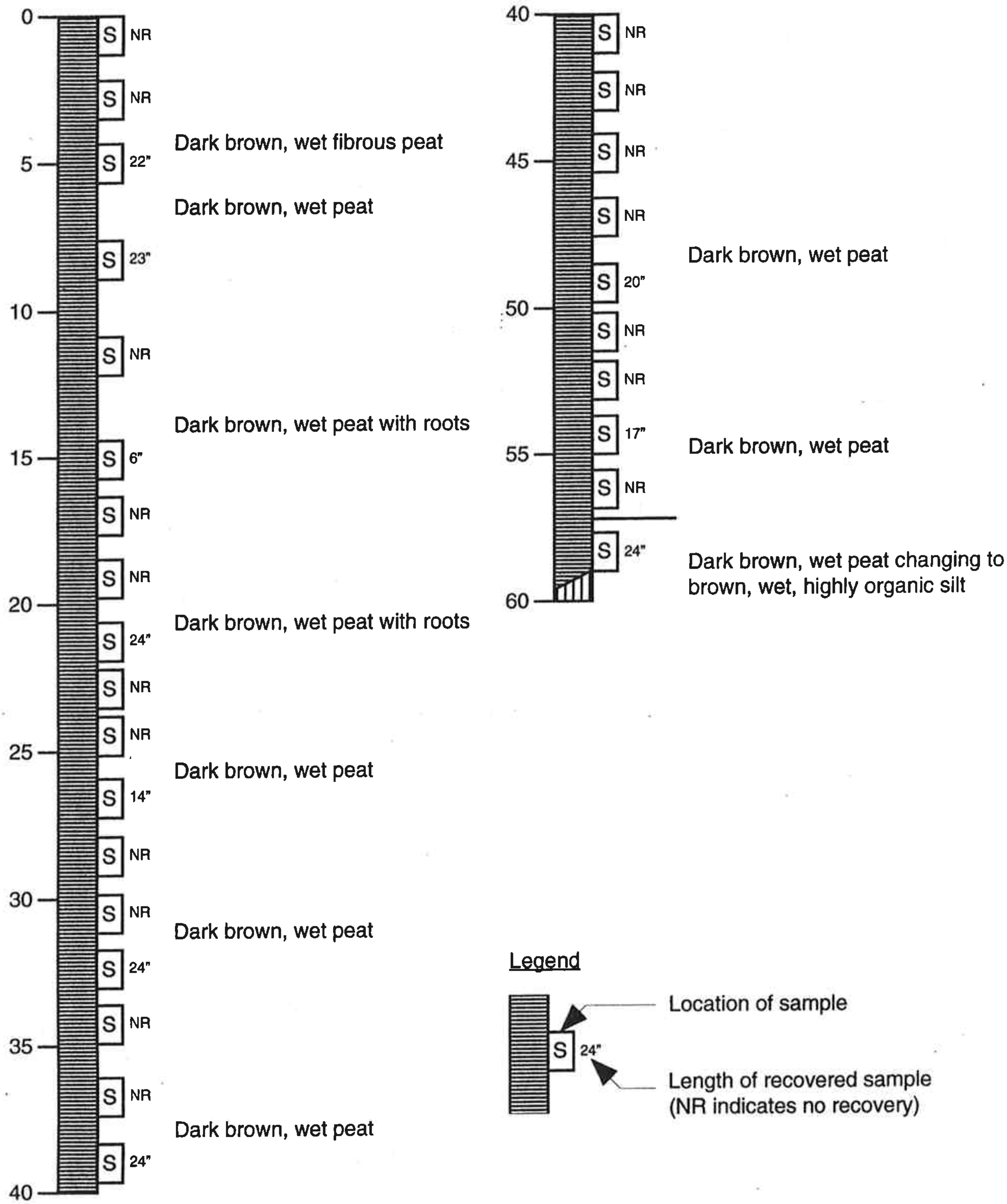


Figure 6. Log of WSDOT Boring in Mercer Slough

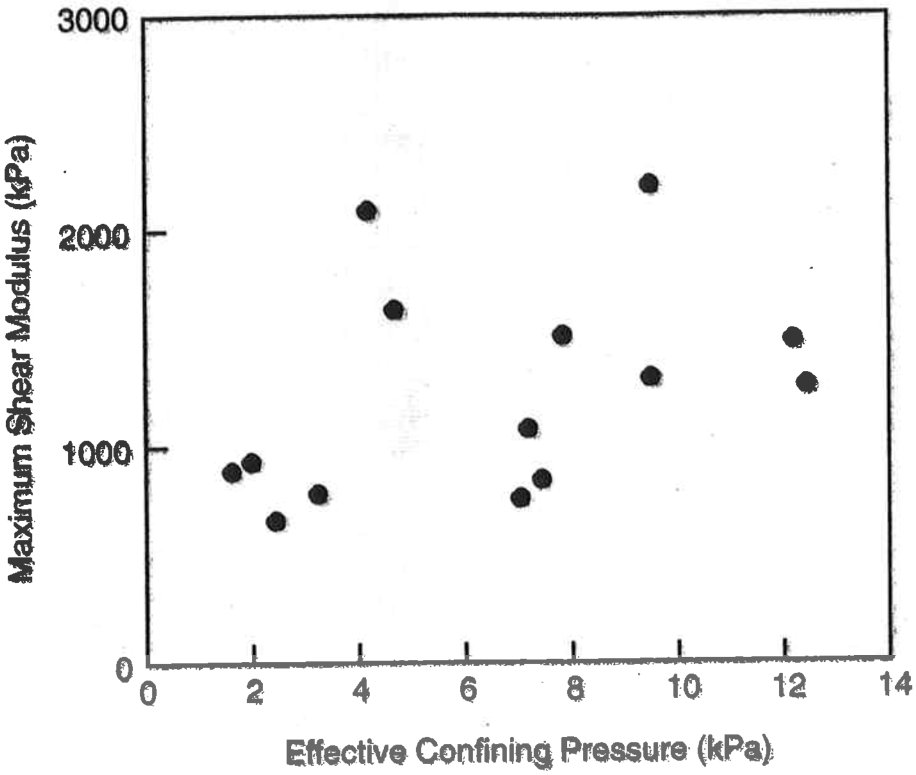


Figure 7. Variation of Maximum Shear Modulus with Effective Confining Pressure

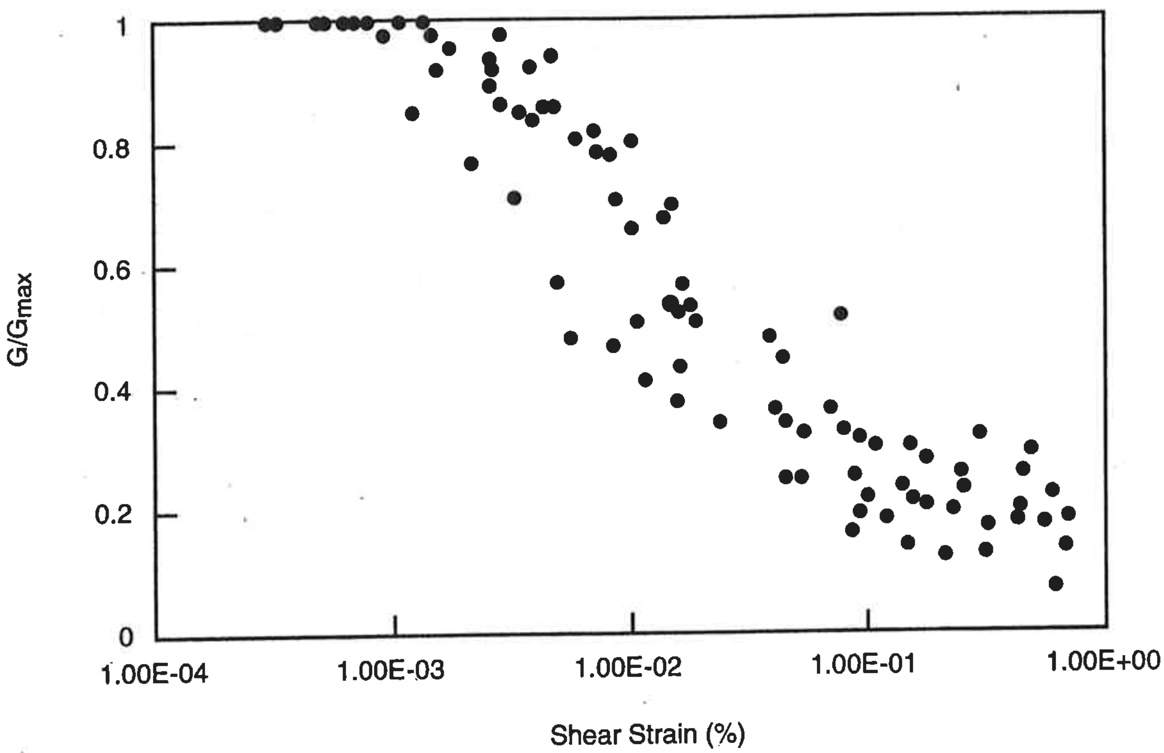


Figure 8. Modulus Reduction Behavior for Normally Consolidated Mercer Slough Peat Specimens at Effective Confining Pressures of 2 kPa to 12.5 kPa

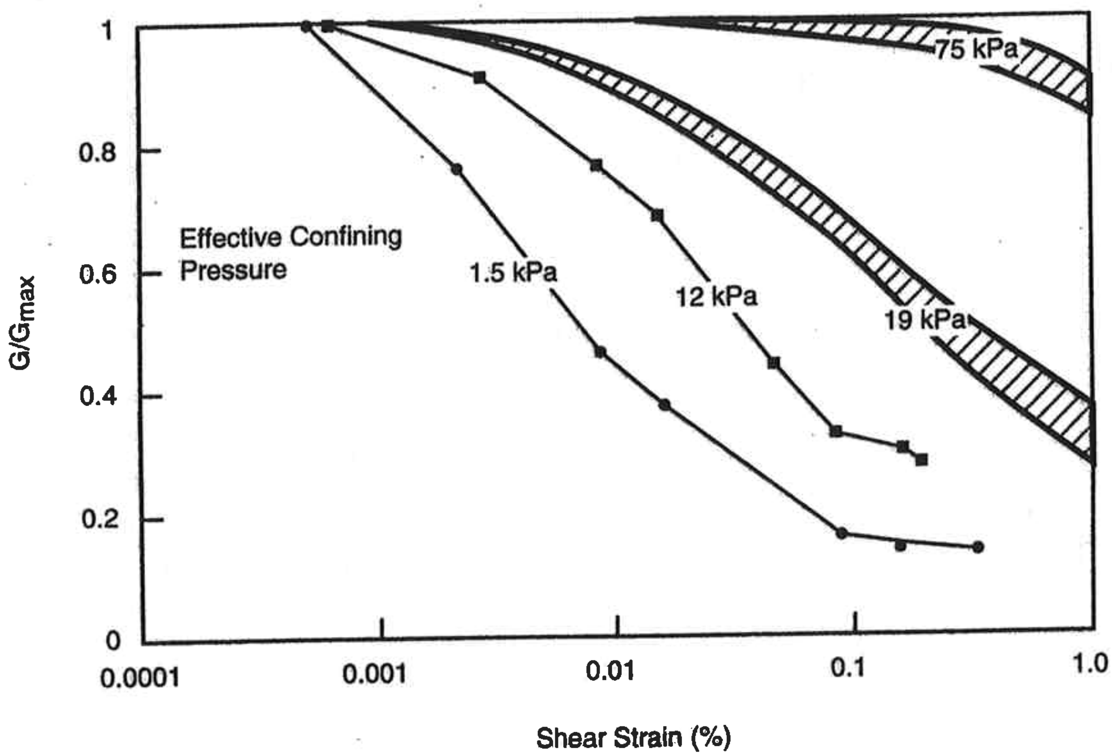


Figure 9. Variation of Modulus Reduction Behavior with Effective Confining Pressure. Curves for effective confining pressures of 1.5 kPa and 12.5 kPa are from resonant column tests conducted in this investigation. Curve for effective confining pressure of 19 kPa is from cyclic triaxial and piezoelectric bender elements tests on Mercer Slough peat by Kramer et al., (1991). Curve for effective confining pressure of 75 kPa is from resonant column and torsional shear tests on peat from eastern United States (K. Stokoe, personal communication, 1996).

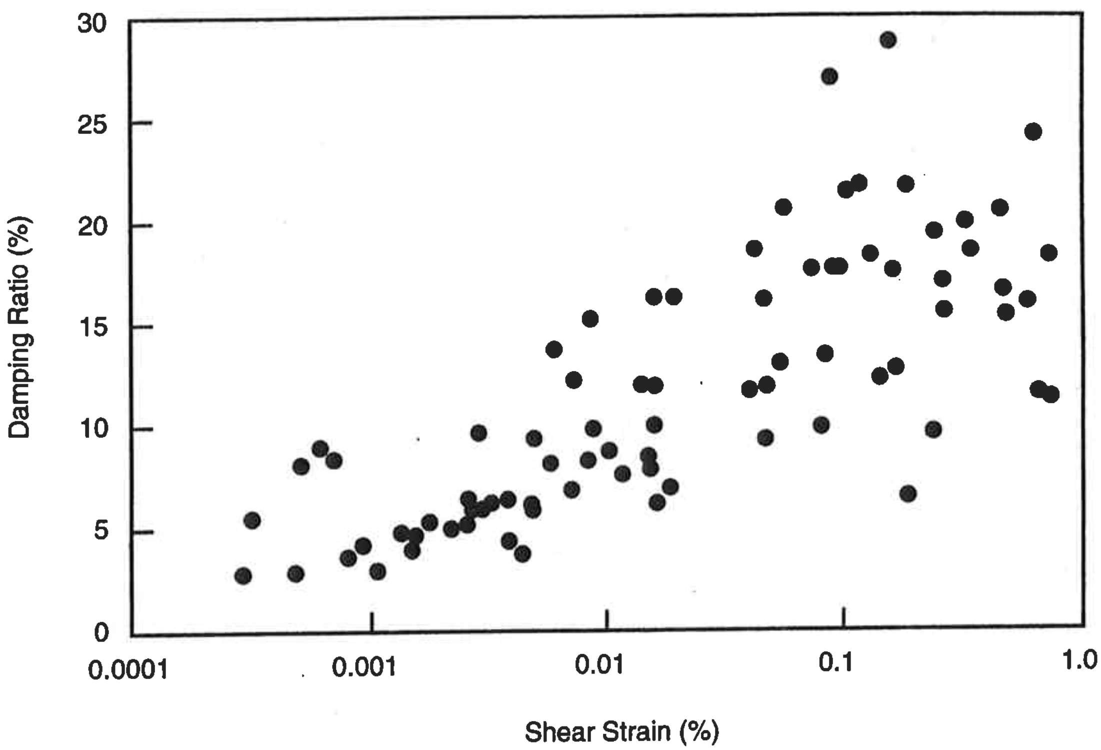


Figure 10. Damping Behavior for Normally Consolidated Mercer Slough Peat Specimens at Effective Confining Pressures of 1.5 kPa to 12.5 kPa.

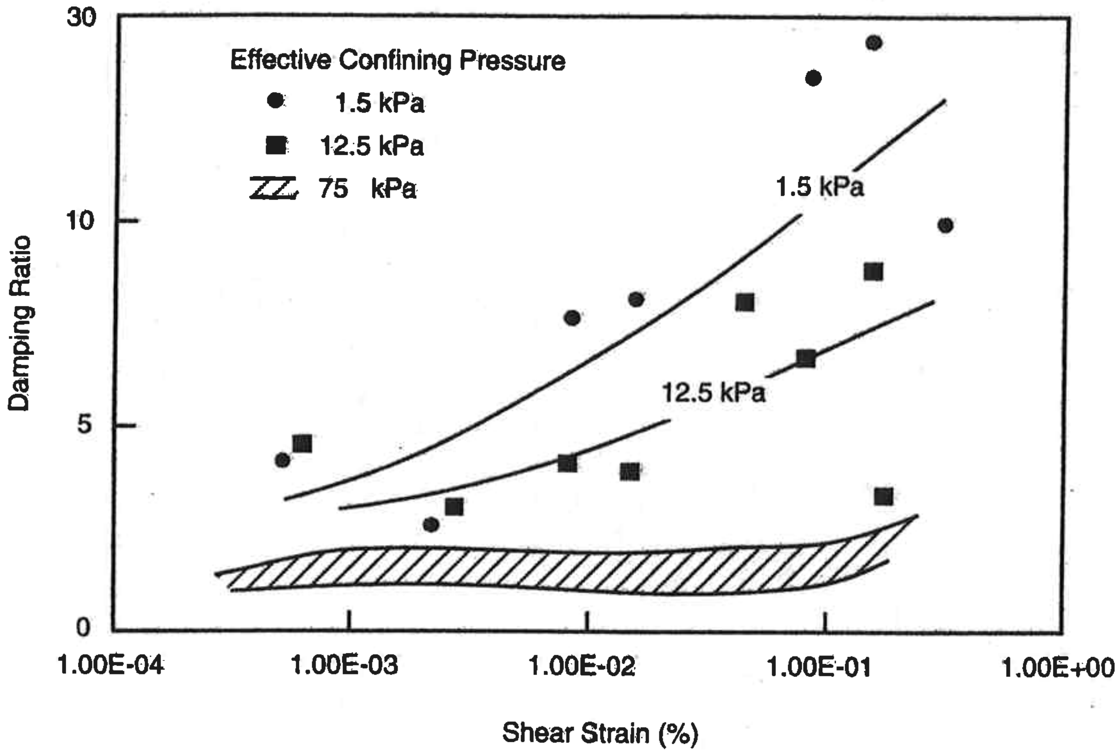


Figure 11. Variation of Modulus Reduction Behavior with Effective Confining Pressure. Curves for effective confining pressures of 1.5 kPa and 12.5 kPa are from resonant column tests conducted in this investigation. Curve for effective confining pressure of 75 kPa is from resonant column and torsional shear tests on peat from eastern United States (K. Stokoe, personal communication, 1996).

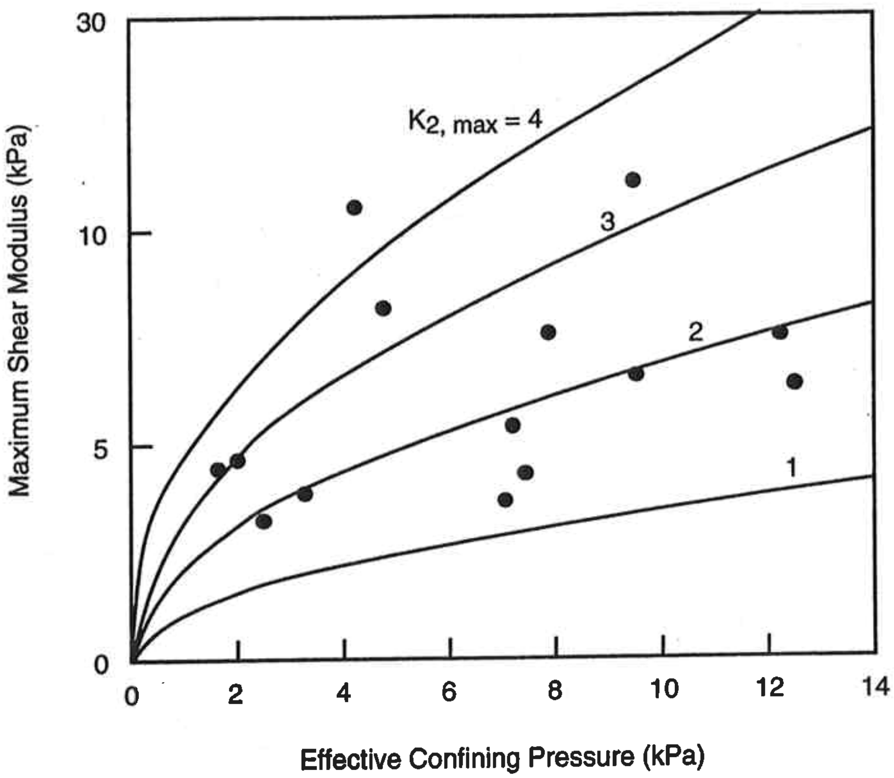


Figure 12. Contours of $K_{2,max}$ for Mercer Slough Peat

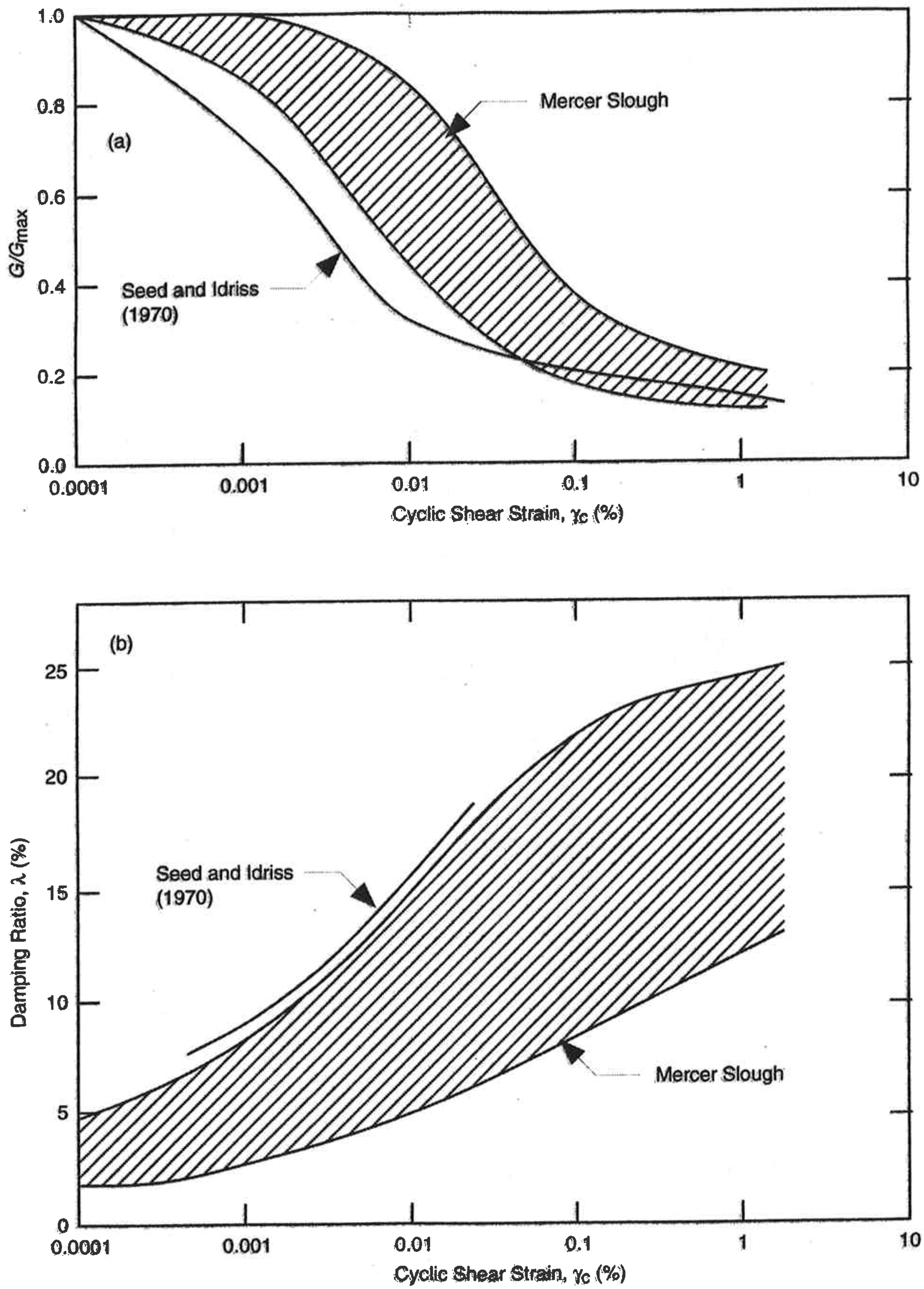


Figure 13. Comparison of (a) Modulus Reduction and (b) Damping Behavior for Tests on Union Bay Peat (Seed and Idriss, 1970) and Mercer Slough Peat.

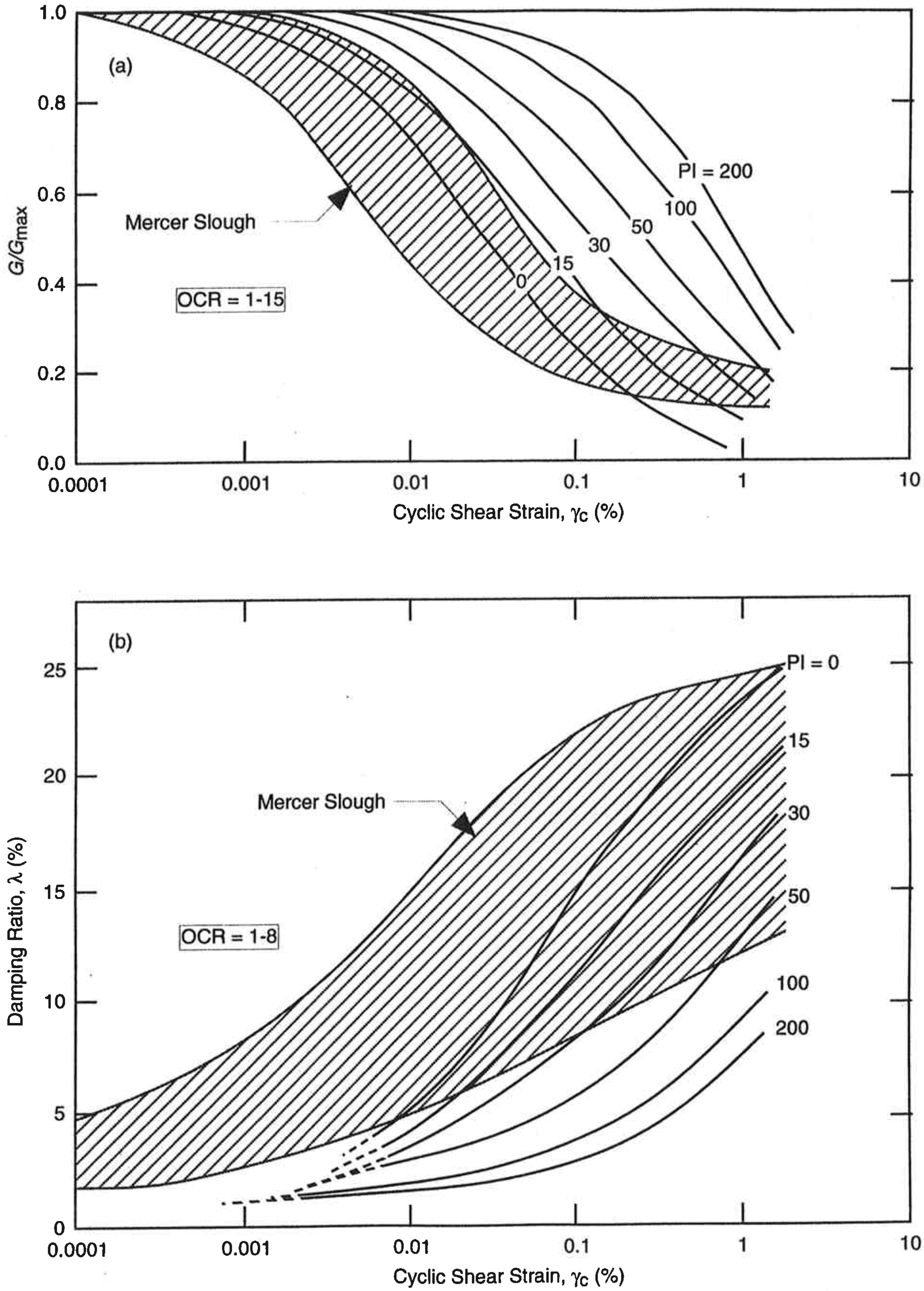


Figure 14. Comparison of (a) Modulus Reduction and (b) Damping Behavior for Mercer Slough Peat and Inorganic Soils.

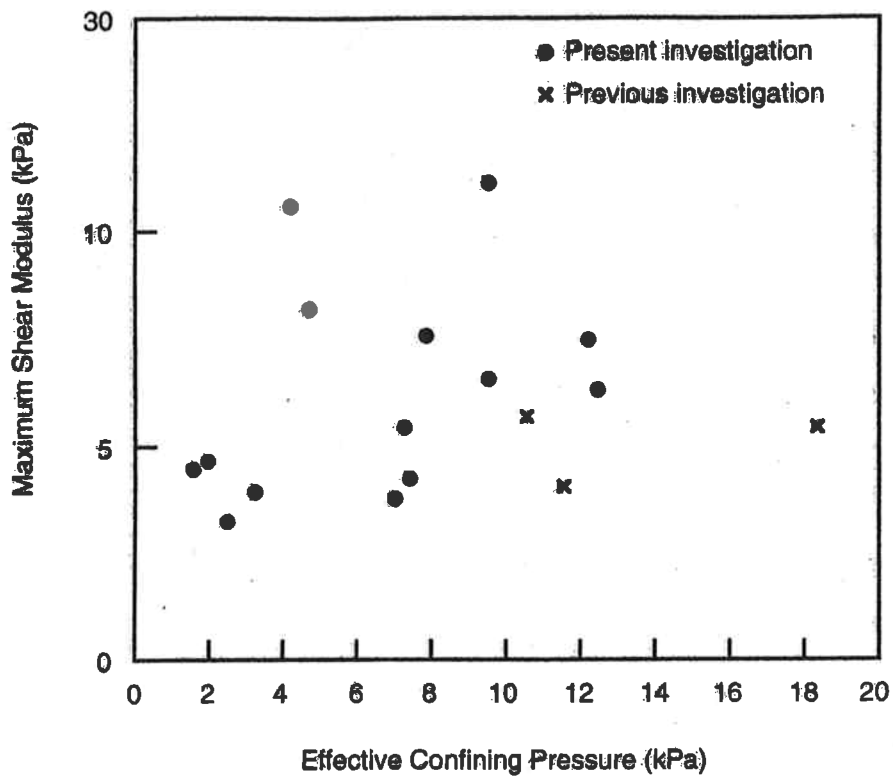


Figure 15. Comparison of Maximum Shear Moduli from Present Investigation and Previous Investigation (Kramer et al., 1991).

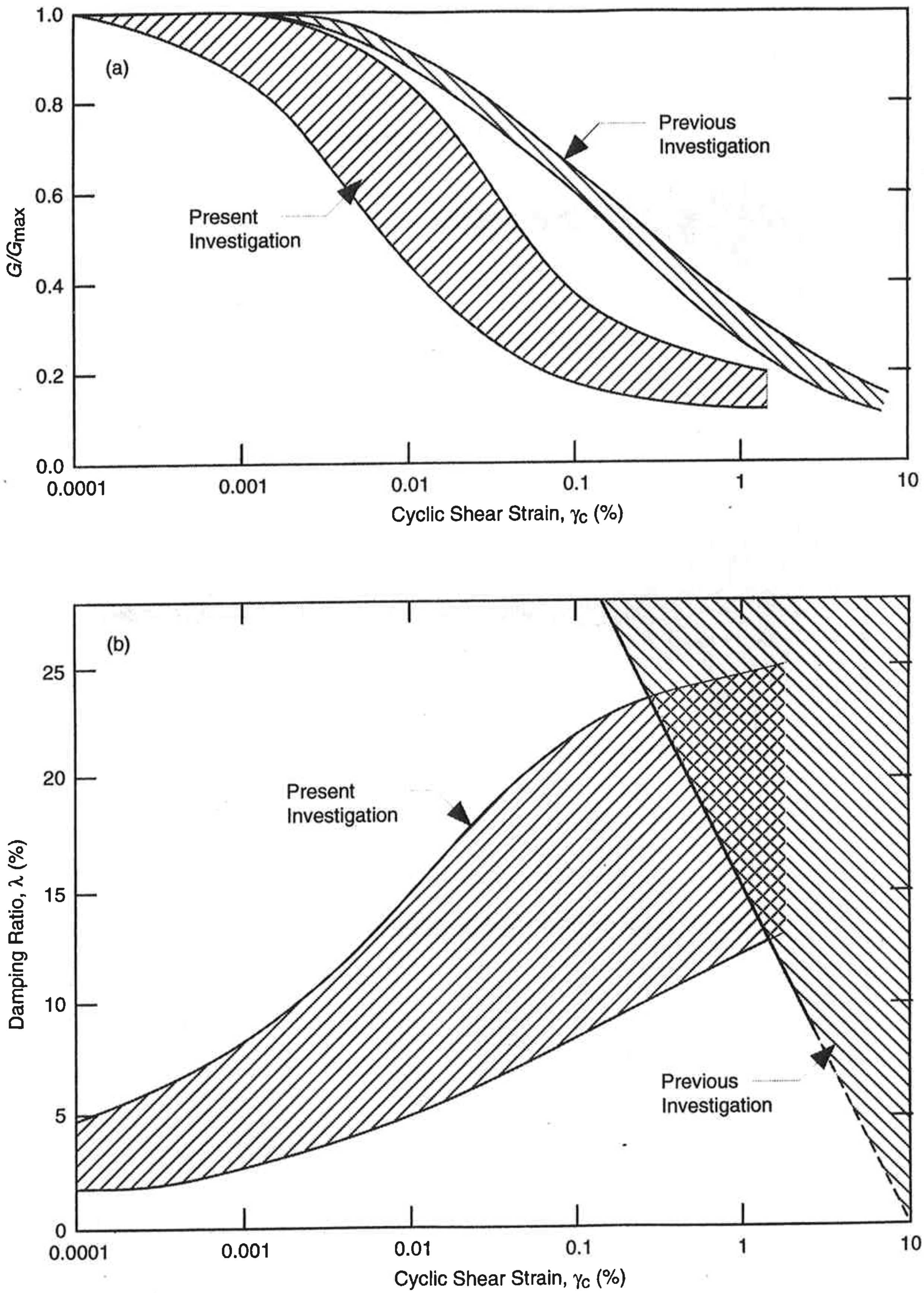
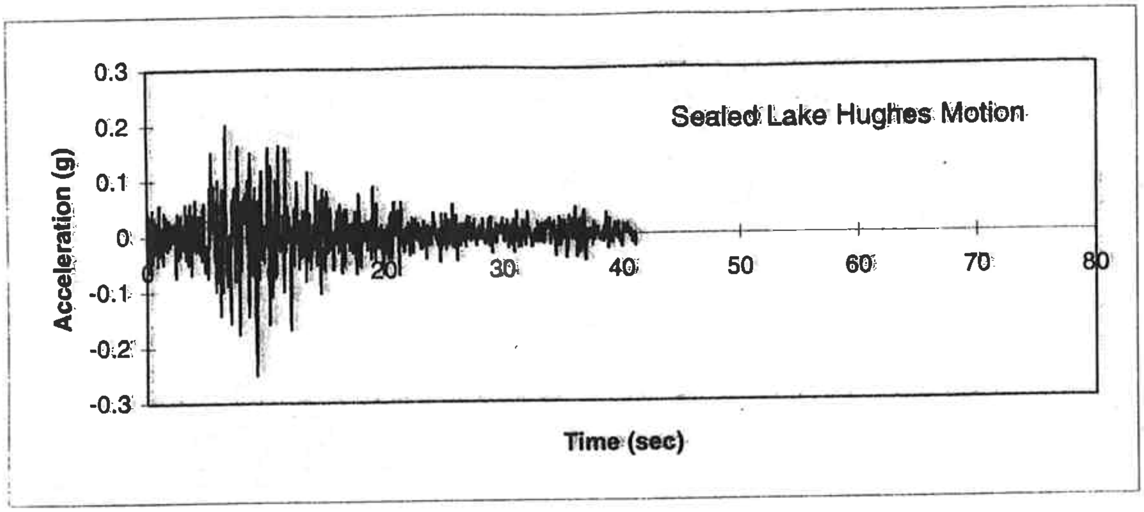


Figure 16. Comparison of (a) Modulus Reduction and (b) Damping Behavior for Present Investigation and Previous Investigation (Kramer et al., 1991).



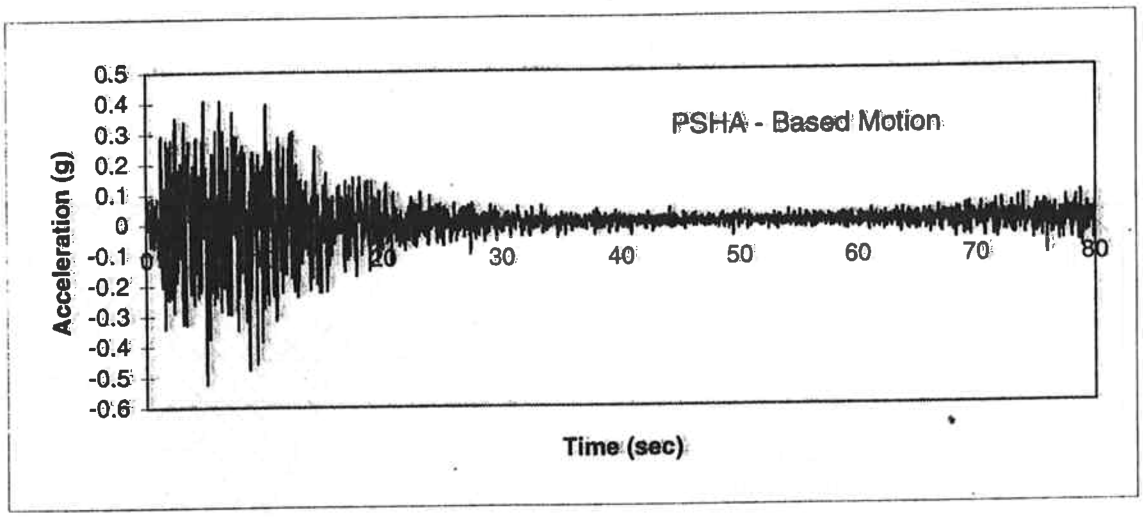


Figure 17. Input Motions Used in Ground Response Analyses.

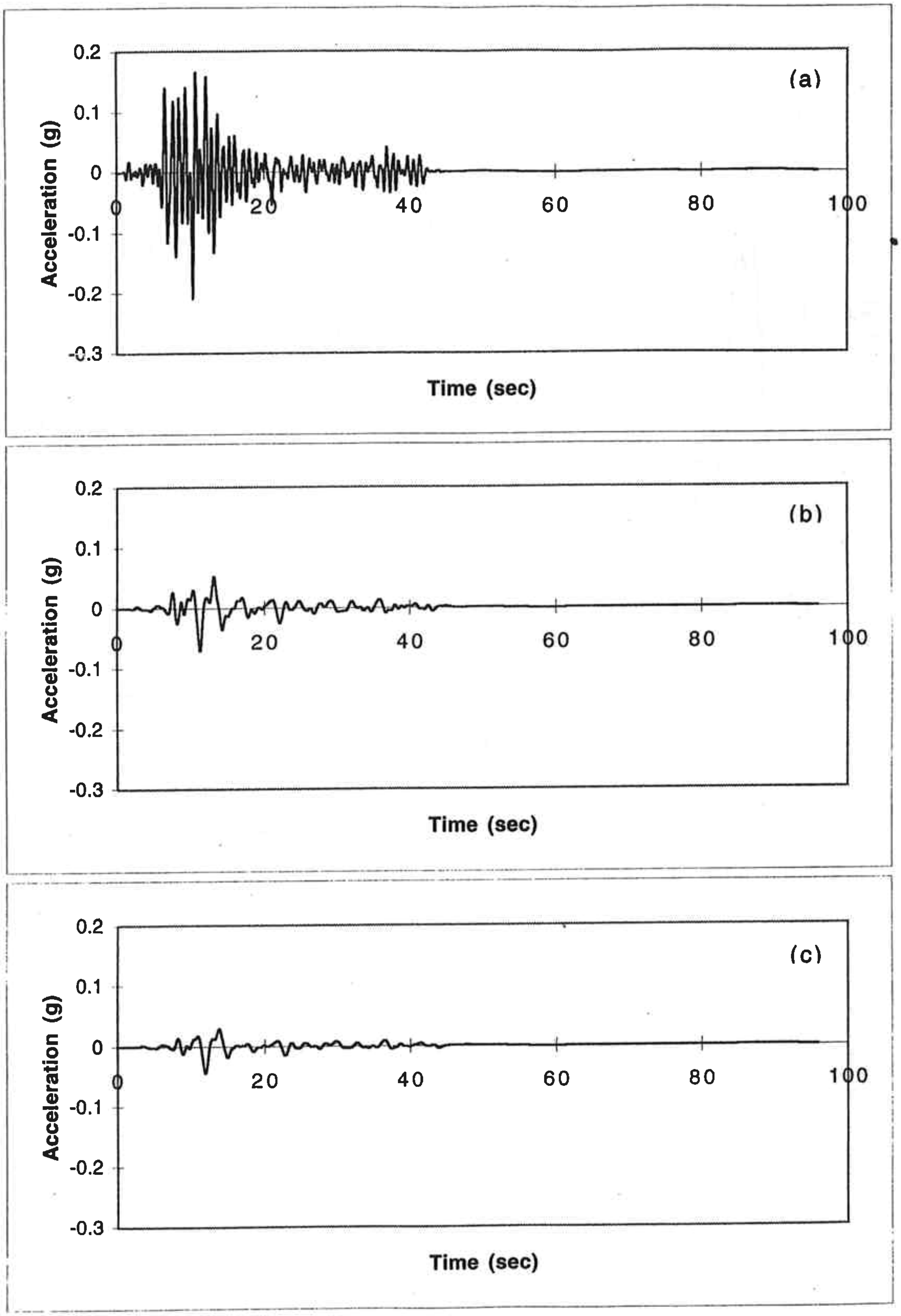


Figure 18. Time Histories of Ground Surface Acceleration for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the Lake Hughes Input Motion. Ground Motions Computed Using Equivalent Linear Ground Response Analysis.

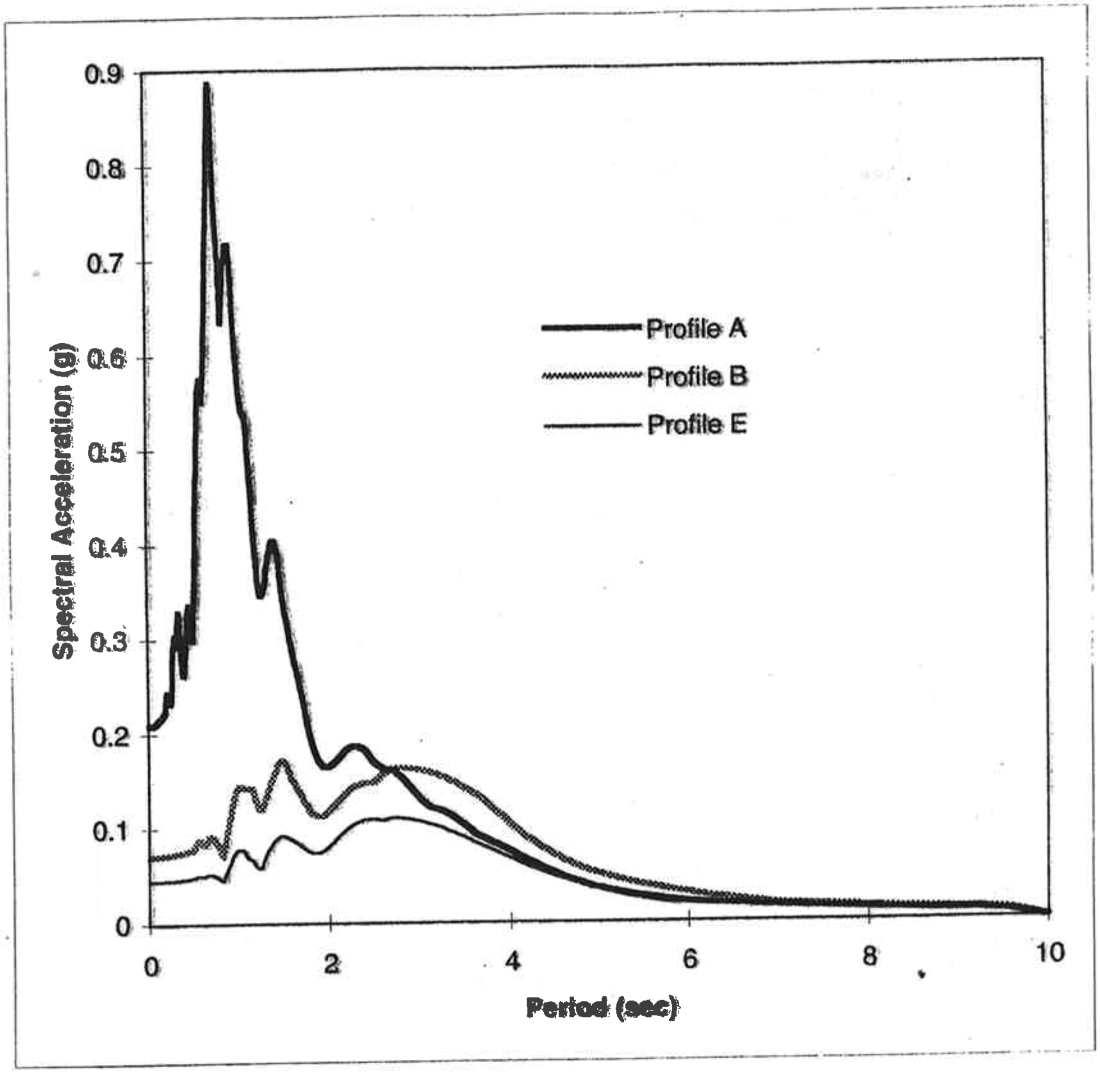


Figure 19. Ground Surface Response Spectra (5% Damping) for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the Lake Hughes Input Motion. Ground Motions Computed Using Equivalent Linear Ground Response Analysis.

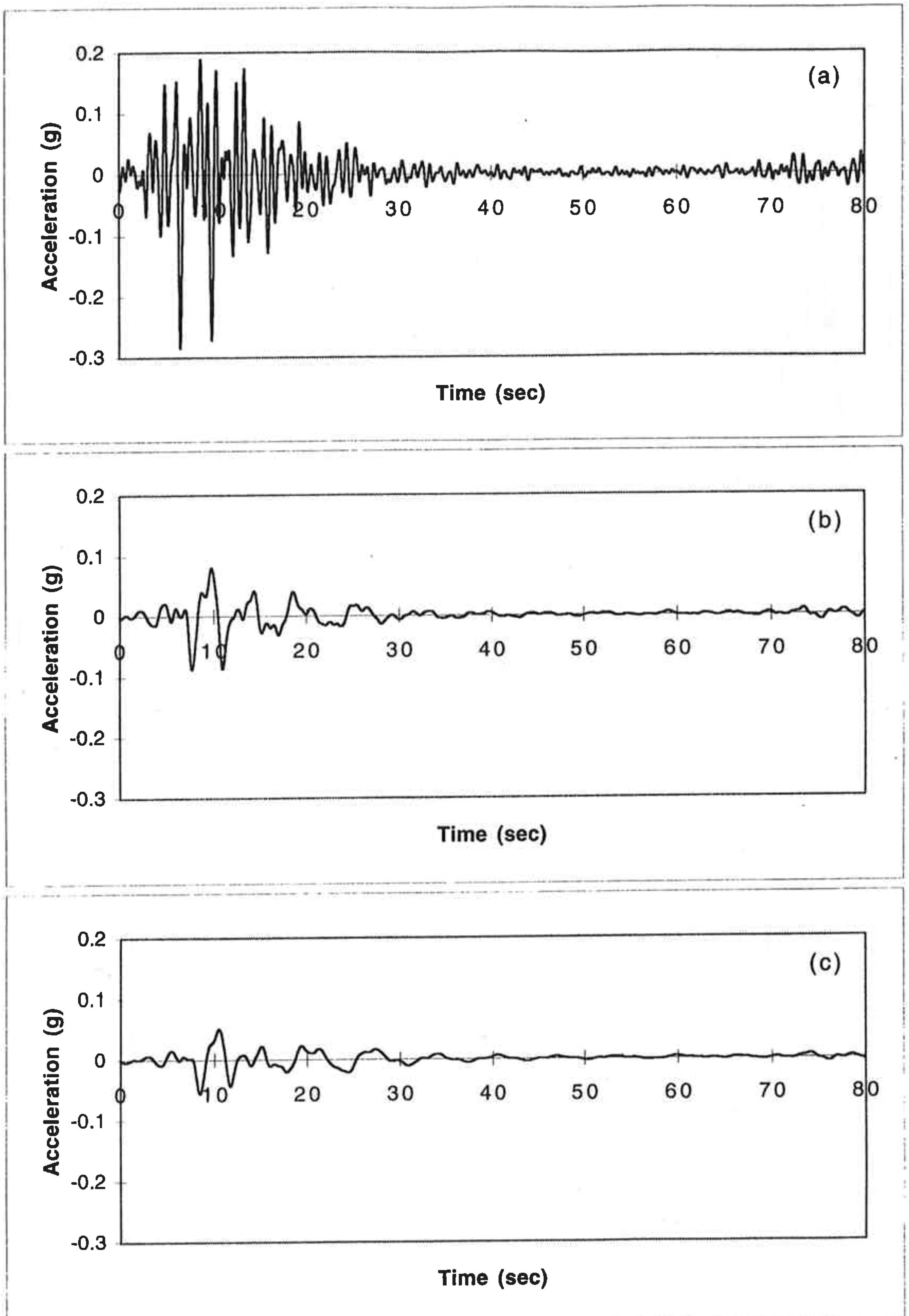


Figure 20. Time Histories of Ground Surface Acceleration for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the PSHA-based Input Motion. Ground Motions Computed Using Equivalent Linear Ground Response Analysis.

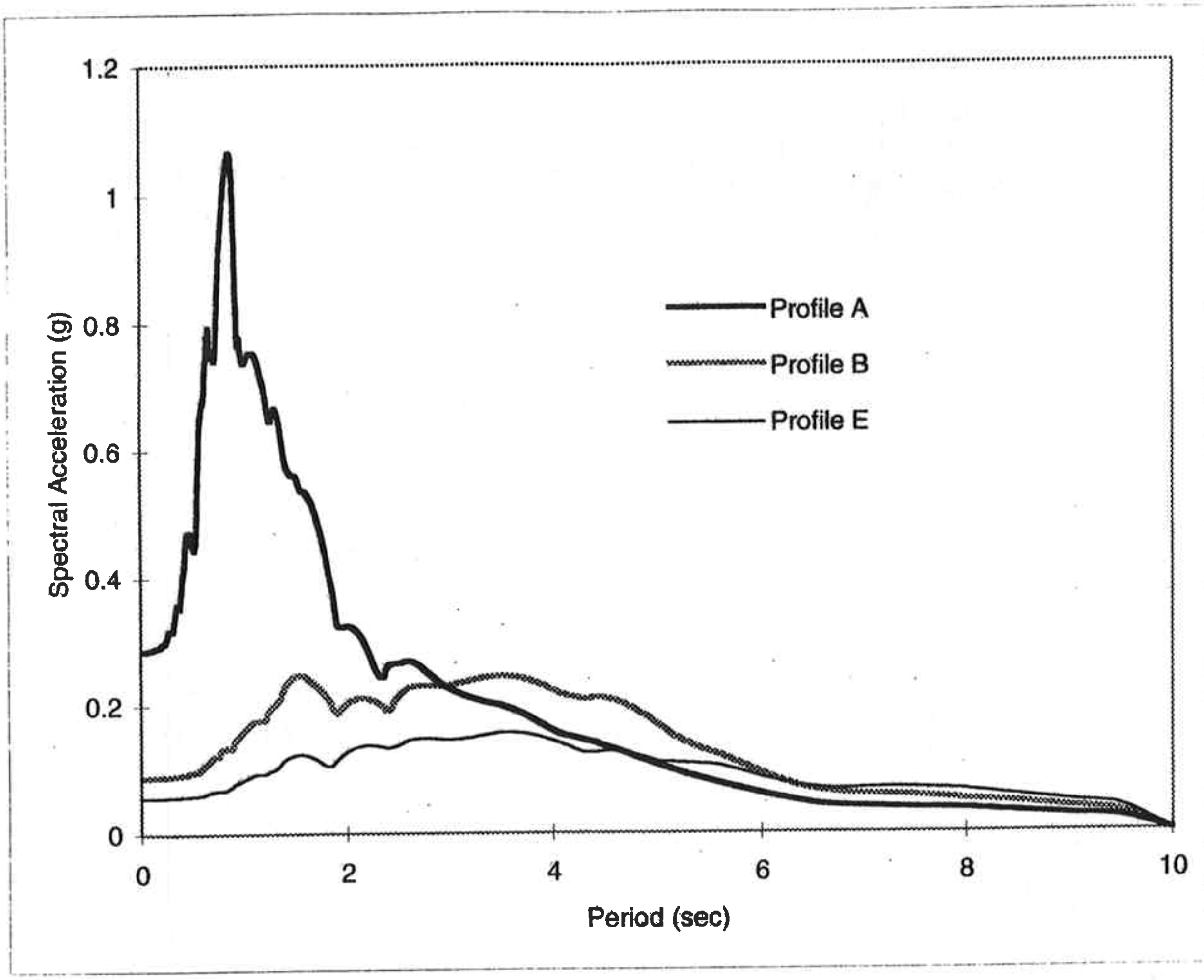


Figure 21. Ground Surface Response Spectra (5% Damping) for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the PSHA-based Input Motion. Ground Motions Computed Using Equivalent Linear Ground Response Analysis.

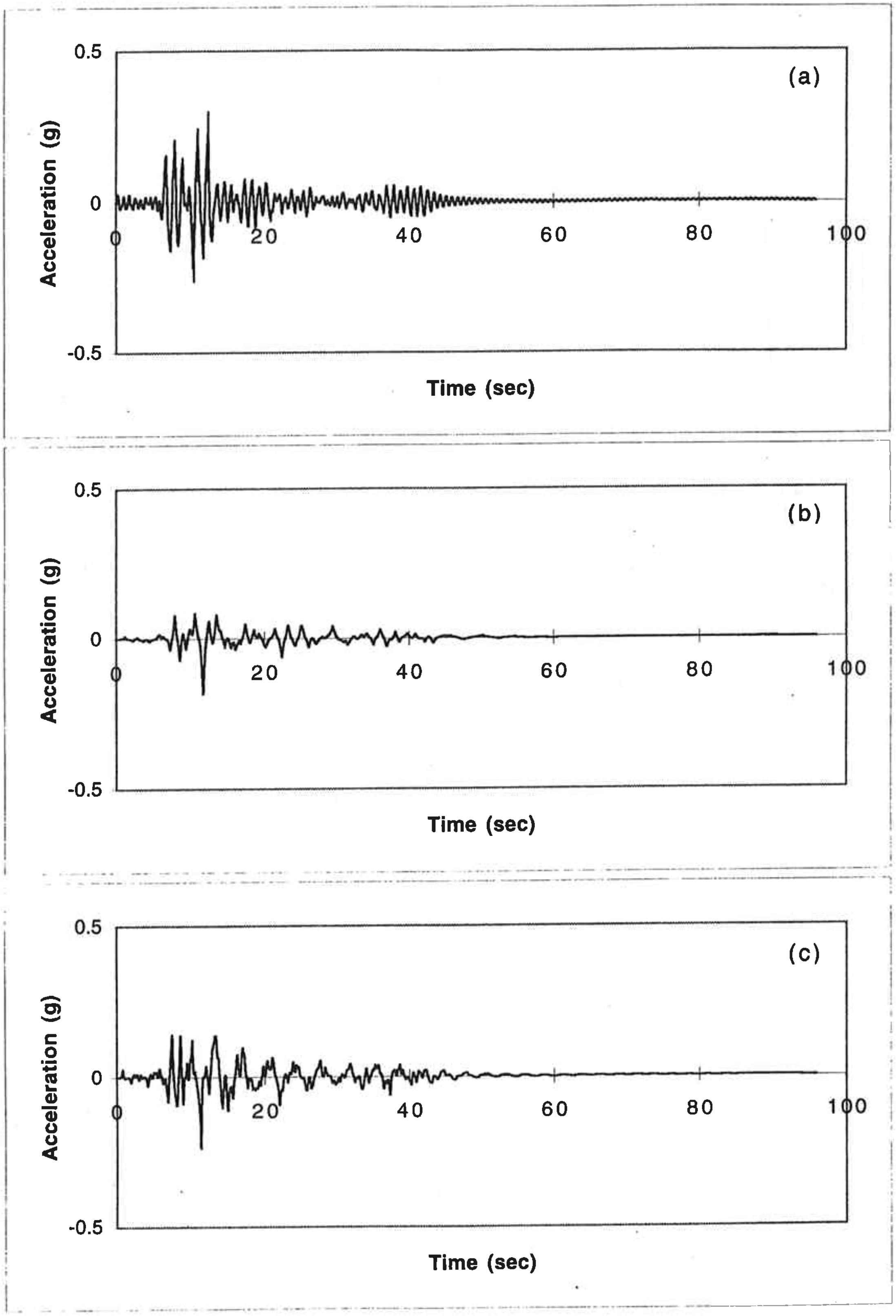


Figure 22. Time Histories of Ground Surface Acceleration for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the Lake Hughes Input Motion. Ground Motions Computed Using Nonlinear Ground Response Analysis.

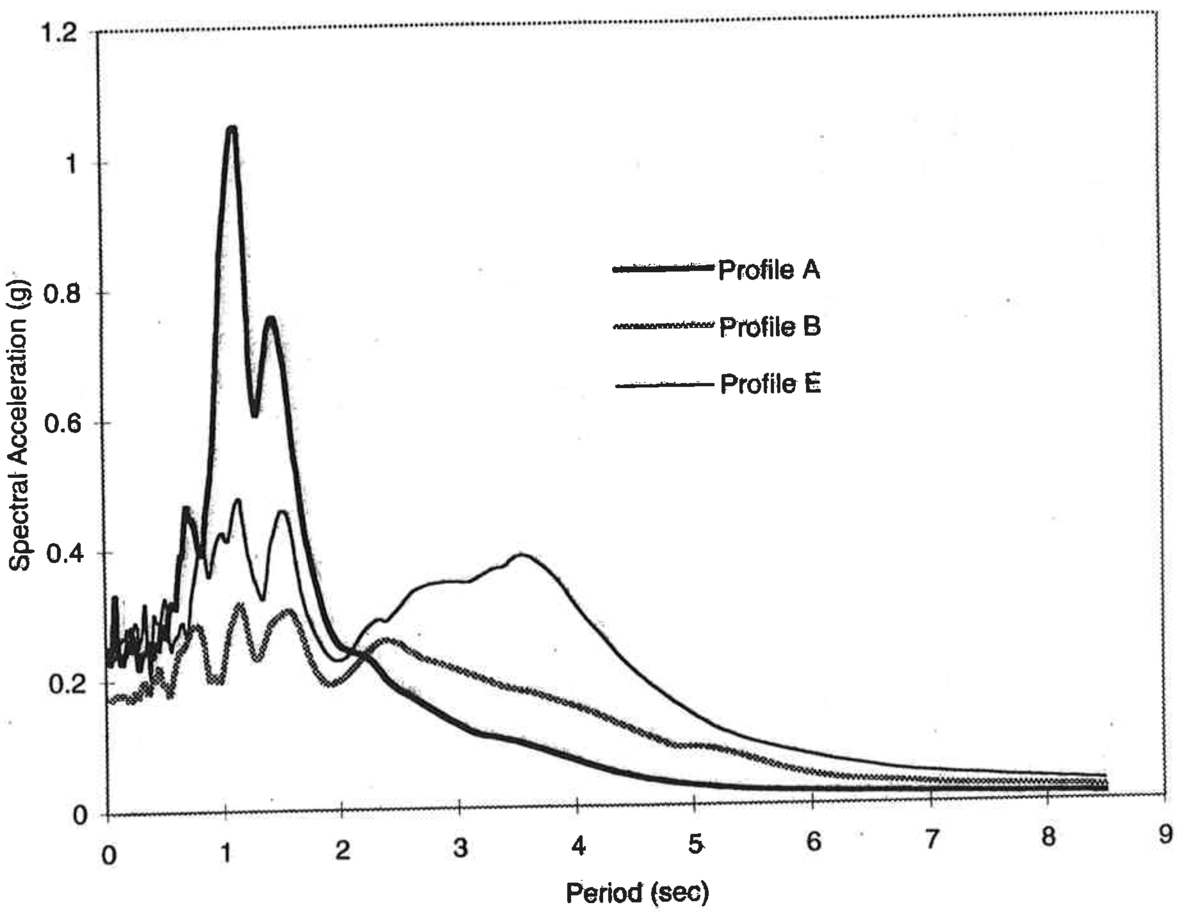


Figure 23. Ground Surface Response Spectra (5% Damping) for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the Lake Hughes Input Motion. Ground Motions Computed Using Nonlinear Ground Response Analysis.

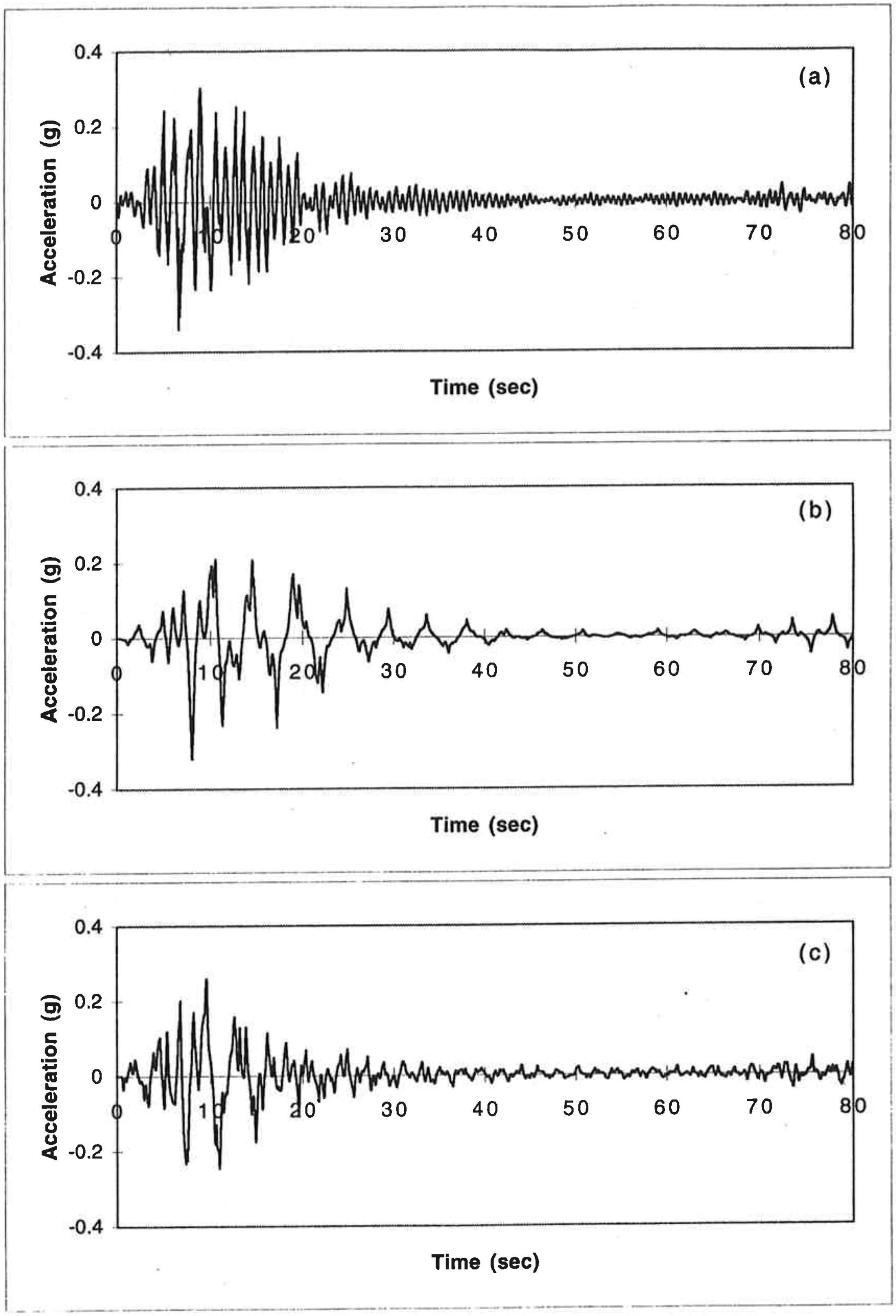


Figure 24. Time Histories of Ground Surface Acceleration for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the PSHA-based Input Motion. Ground Motions Computed Using Nonlinear Ground Response Analysis.

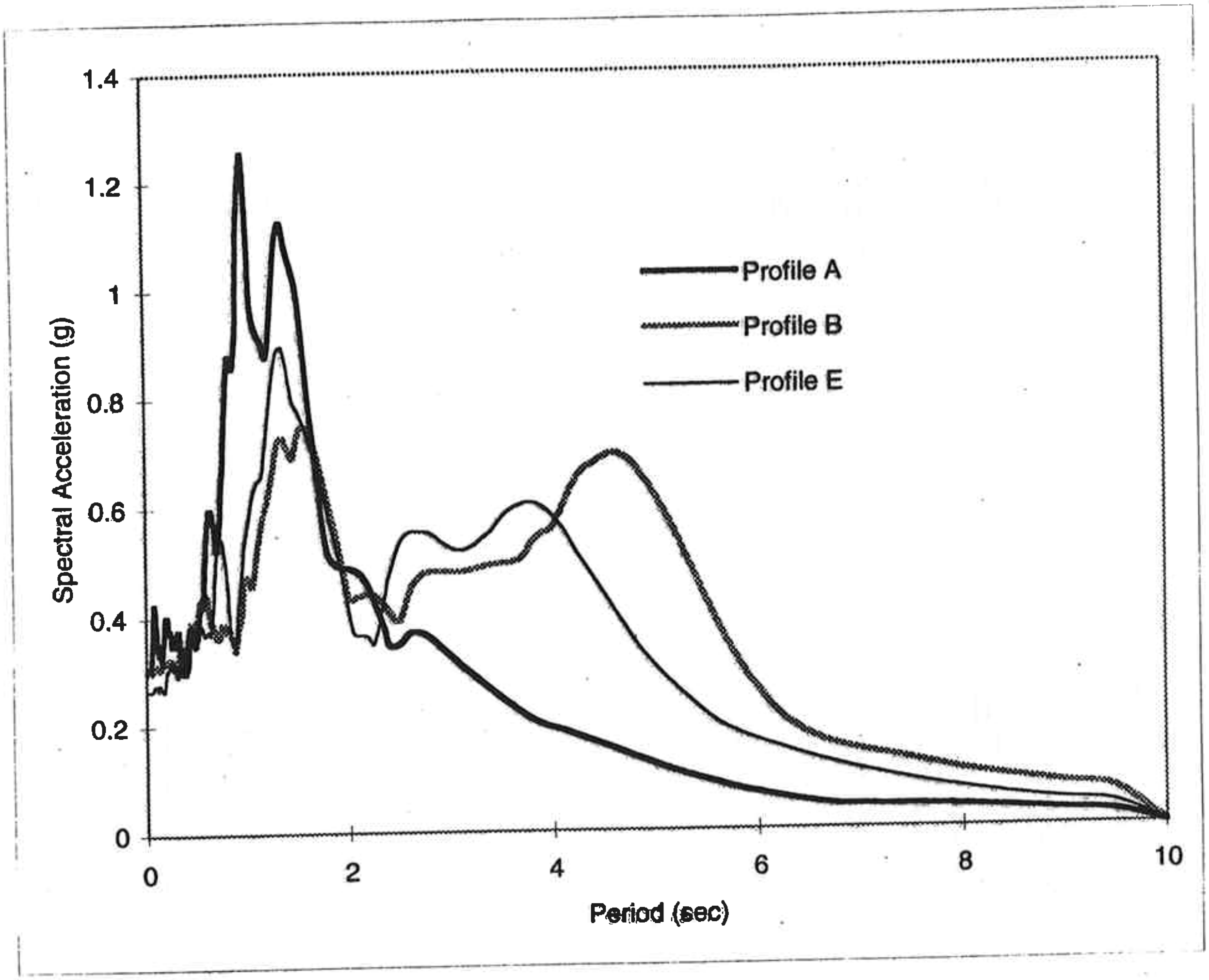


Figure 25. Ground Surface Response Spectra (5% Damping) for (a) Profile A, (b) Profile B, and (c) Profile E Subjected to the PSHA-based Input Motion. Ground Motions Computed Using Nonlinear Ground Response Analysis.

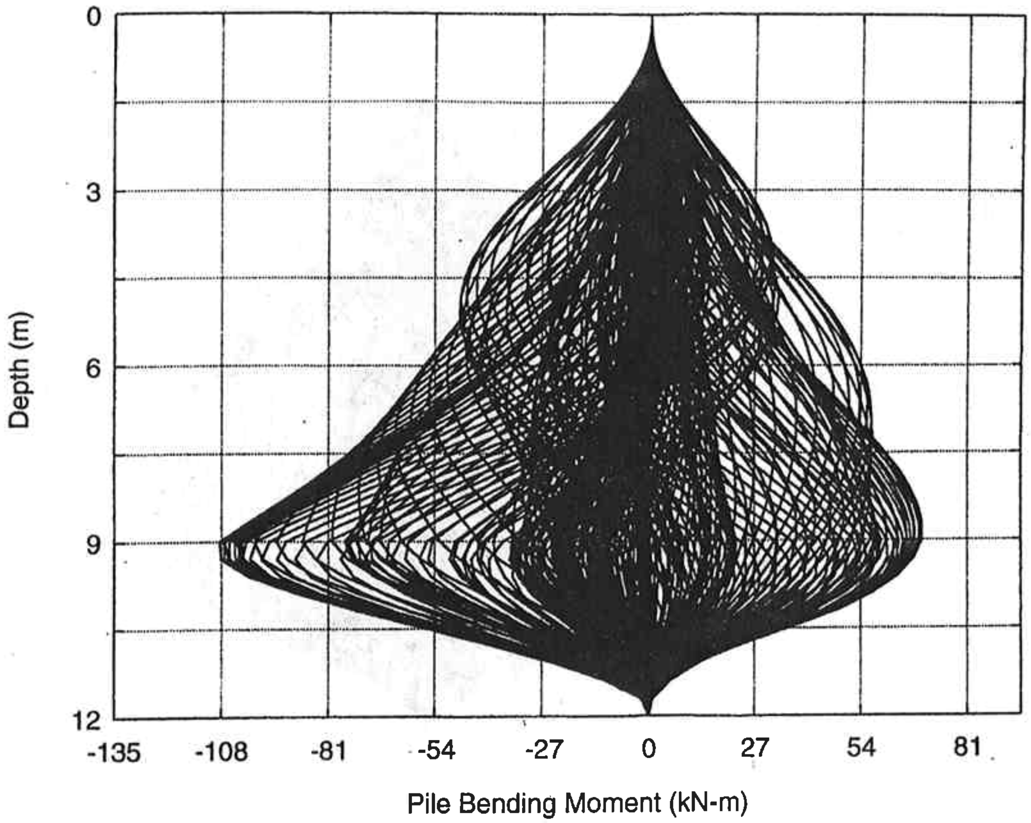


Figure 26. Bending Moment Profiles for 30.5-cm-diameter pile at Section B Subjected to the Lake Hughes Input Motion.

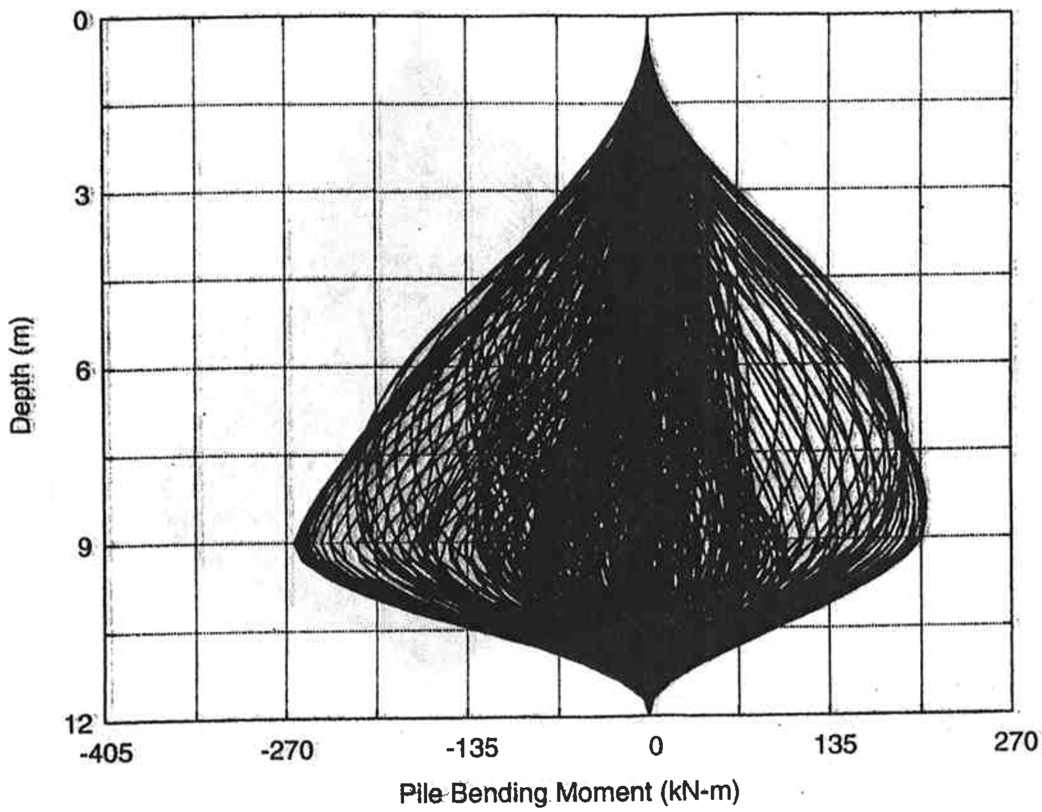


Figure 27. Bending Moment Profiles for 61-cm-diameter Pile at Section B Subjected to the Lake Hughes Input Motion.

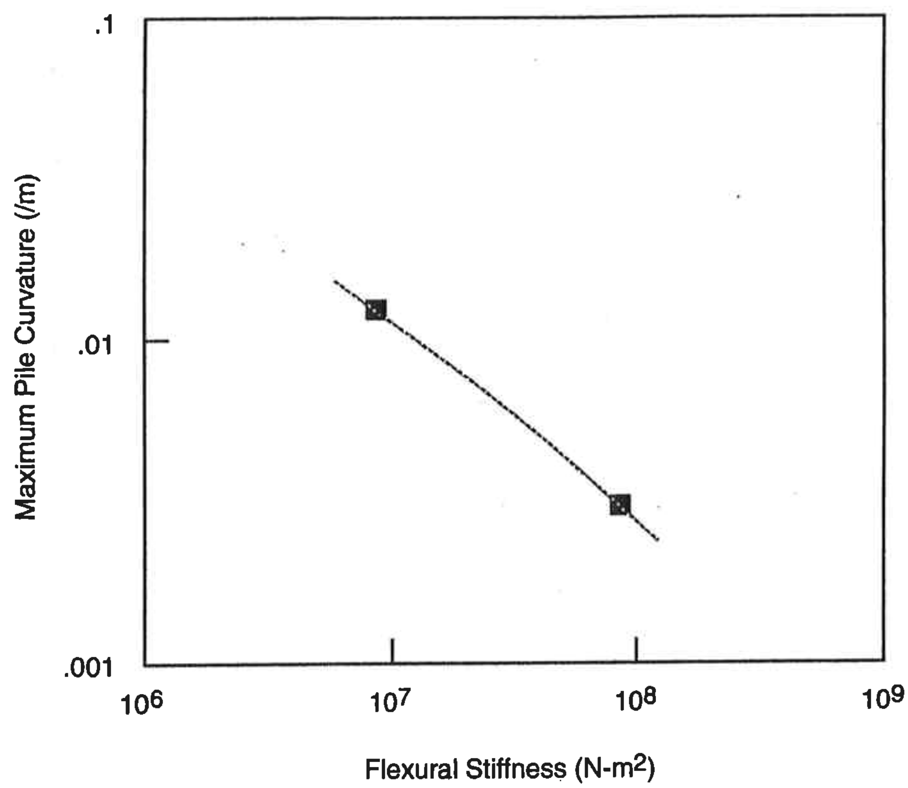


Figure 28. Estimation Variation of Maximum Pile Curvature with Flexural Stiffness for Approximate Estimation of Maximum Bending Moments of Piles at Section B Subjected to the Lake Hughes Input Motion.